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# 30% DESIGN
# SOIL AND FOUNDATION INVESTIGATION

## Proposed
## 6th Avenue Freeway over BNSF Bridge Replacement
## City and County of Denver, Colorado

**Prepared For**

**Wilson & Company, Inc.**
**Attn: Scott Waterman, P.E.**
**999 18th Street, Suite 2600**
**Denver, Colorado 80202**

**December 19, 2011**
**Revised March 9, 2012**

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Prepared For

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# TABLE OF CONTENTS

Page



## LIST OF FIGURES, TABLES AND APPENDIXES

# 1.0 PURPOSE AND SCOPE

This 30% design level report contains the results of a preliminary soil and foundation investigation conducted for the proposed replacement of the existing three-span 6th Avenue Bridge over Burlington Northern Santa Fe (BNSF) Railroad in Denver, Colorado. The project is being conducted under the Colorado Bridge Enterprise Program administered under the direction of the Colorado Department of Transportation (CDOT). Affected rights-of-way are controlled by City and County of Denver, BNSF Railroad, and CDOT.

A field subsurface investigation was conducted to obtain information on pavement, soil, bedrock, and ground water conditions. Soil and bedrock samples were visually classified, and selected samples were laboratory tested to evaluate strength, compressibility or swell characteristics, classification, and chemical properties. The results of the field and laboratory investigations were analyzed to develop preliminary recommendations for foundations, retaining walls, and pavements. We understand that this project will be continued under a design/build procedure and that the design/build contractor will be responsible for final design. The investigation was conducted in general accordance with our Subconsultant Agreement/Subcontract No. 001 (WCI File No. 11-100-30102) with Wilson & Co., Inc, dated March 11, 2011. The investigation is identified by CDOT as "Task Order #2 "Preliminary Design of 6th Ave Bridge over BNSF Railroad".

This report has been prepared to summarize the data obtained and to present our preliminary conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed structures are included. Limited environmental monitoring and sampling was conducted by Pinyon Environmental during Geocal's drilling operations. More specific investigations related to possible hazardous materials are beyond the scope of this study.

# 2.0 PROPOSED CONSTRUCTION

The proposed construction is expected to consist principally of replacing the existing bridge with a similar two-span structure with abutments and piers to be near their current locations. New retaining walls will likely be cast-in-place cantilevered or possibly soil nail. Changes to the approach embankment fills and new deck alignment are expected to be minor. New pavements may be constructed for the (bridge) approaches, but grades are expected to be similar to existing.

# 3.0 SITE CONDITIONS

The project site is situated on the transition between lower terraces of the eastern pre-controlled floodplain of the South Platte River and upper terraces associated with slightly elevated ground (sometimes refered to as Lincoln Park Uplands) between the South Platte River and Cherry Creek Valley. The original natural terraces have been modified during the development of transportation, industrial-commercial, and drainage control projects in the area. The north-flowing South Platte River is about one-half mile west of the site and separated from the site by the 6th Avenue/I-25 Interchange. 6th Avenue within the project area is elevated on constructed embankments and bridge structures, continuously from west of the South Platte to North Klamath Street on the east.

The bridge crosses over a rail corridor having two mainline and two siding heavy rail tracks. Tracks are all under the wider west span with rail beds about 25 feet below bottom of girders. The east span is underlain by steep concrete slope pavement extending from near track level to just below the bridge girders. Embankment side slopes near the bridge are covered with sparse grass, weeds, brush and scattered deciduous trees. Areas east of the bridge are occupied by light industrial and warehouse-type businesses in low-rise structures; land to the west is essentially dedicated to I-25 right-of-way.

# 4.0 SITE GEOLOGY

Standard quadrangle-scale published geologic mapping indicates that natural (pre-construction) unconsolidated surficial and shallow deposits include:

1. Near river floodplain soil assigned to the Post-Piney Creek Alluvium generally as interbeds and mixtures of humic clay, silt, sand, and occasional small gravel. Thicknesses of 5 feet to 10 feet are typical (where not removed by construction). Local, but significantly thick lenses of highly humic bog clays and silt have been noted. Mapping indicates Post-Piney Creek soil covered the surface west of the current track corridor.

2. The upper floodplain terrace soil identified as Piney Creek Alluvium and typified as well stratified clay, silt and sand (including mixtures of) that are commonly humic in the uppermost and gravelly near the base. The Piney Creek Alluvium has been reported as 5 feet to 10 feet thick, mapped as originally covering the surface east of the tracks, and indicated as extending under portions of Post-Piney Creek deposits.

3. Older upper terrace deposits assigned to Broadway Alluvium as moderately well-graded sand and gravel with generally limited fines. These deposits are mapped on higher terraces east and west of the South Platte River and interpreted as commonly extending under Piney Creek and Post-Piney Creek soils in the project area.

The above soil deposits are indicated to lie on well-stratified sedimentary bedrock assigned to the Denver-Arapahoe Formations (undifferentiated). At depths associated with potential construction, members of the formations are typically dominated by claystone and siltstone interbeds with lesser interbeds and lenses of sandstone. Outcrops or construction excavated exposures of this material are mapped within a mile of the project site, and are refered to in published reports of nearby soil borings and water wells. Published mapping indicates bedrock to have about 20 feet of natural alluvium cover (excluding embankment fills) in the vicinity of the bridge and to be flat to very gently dipping.

# 5.0 SUBSURFACE INVESTIGATION

The subsurface investigation for this project included drilling eight exploratory borings from October 31, 2011 through February 24, 2012 at the approximate locations shown on Figure 1, Locations of Exploratory Borings. Initial borings (Borings 1 through 6), were advanced with a truck-mounted CME-75 drill rig equipped with 3¼ inch inside diameter (ID) hollow stem augers. Two borings (7 and 8) were drilled at approximate track level, within the BNSF right-of-way and were advanced with a truck-mounted CME-550 equipped with 3¼ inch inside diameter hollow stem augers. All borings were logged by a representative of Geocal. Subsurface soil and bedrock samples were obtained using 2 inch ID California liner samplers and 1³/₈ inch ID split-spoon (Standard Penetration Tester) samplers. The samplers were driven into the various strata with blows from a 140 pound hammer, similar to ASTM D1586 test standard. Penetration resistance values when properly evaluated indicate the relative consistency or density of the soils, or hardness of bedrock. Drive samples were taken at approximately 5 foot to 10 foot intervals. Larger bulk samples of auger cuttings were collected from about the upper 1 foot to 10 feet of selected borings.

Logs of the conditions encountered are shown on Figure 2, Logs of Exploratory Borings. Description of the materials and symbols used on the logs are presented on Figure 3, Legend and Notes for Exploratory Borings.

During drilling of portions of Borings 1 and 6, a representative of Pinyon Environmental, Inc. (Pinyon) conducted limited environmental hazmat monitoring and sampling. While drilling approximately 15 feet above and below groundwater level in these borings, open hole air, auger cuttings, and drive samples were monitored for total organic compounds and explosive limits using a field-portable photo ionization detector. Additionally, bailed samples of groundwater were collected once groundwater was encountered. in the borings. The results of the field and laboratory investigations performed by Pinyon are reported elsewhere.

# 6.0 SUBSURFACE CONDITIONS

As shown on the Figure 2, the subsurface conditions varied slightly. In general, the borings encountered relatively thick sections of man-placed embankment fill (artificial fill) followed by natural mostly granular soils over sedimentary (claystone) bedrock. Five of the six borings drilled from the 6th Avenue street level were drilled through roadway or shoulder pavement consisting of 7 inches to 10 inches of asphalt; no specifically identified aggregate base course material was encountered. Boring 1 was drilled in off-road right-of-way (near the northwest corner of the bridge), and Borings 7 and 8 were drilled in sparse grass-covered BNSF right-of-way, near track level.

Borings 1 through 6 encountered man-placed embankment fill to depths of about 35 feet to 39 feet deep. The fill generally consisted of loose to medium dense slightly clayey to silty sand to gravelly sand that included medium stiff to stiff sandy clay. The fill was generally medium to coarse grained, with small to large gravel, and had low to high plasticity for the clay portions, was moist, and light to dark brown. Asphalt, construction debris, and pieces of glass were found in the lower portions of the fill in some of the borings. Borings 7 and 8, encountered artificial fill at the surface to depths of about 3 feet to 8 feet deep. The fill consisted of clay with silt, sand, and trace gravel, that was medium stiff, had low plasticity, and was moist.

Below the fill, the borings encountered natural medium dense (with some loose and very dense zones) gravel with sand, silt and some clayey zones. In Borings 1, 7, and 8, medium dense sands with some gravel were encountered and silt with sand and some organics was encountered in Boring 6. The gravel was small to medium sized and was rounded to sub-rounded.

In Borings 1 through 6, sedimentary bedrock was encountered from depths of about 48 feet to 53 feet, and extended to the maximum depth explored, 85 feet. Bedrock was encountered at 20 feet and 23½ feet in Borings 7 and 8. The bedrock was comprised of claystone that was very hard, had medium to high plasticity, contained varying amounts of silt and fine grained sand, was moist, and blue to dark grey. The claystone contained some small interbedded lenses of sandstone.

Ground water was measured between about 32 feet and 40 feet in Borings 1 through 6 immediately after drilling. Ground water was measured at about 10 feet and 11 feet deep immediately after drilling Borings 7 and 8. The ground water level had changed little after 1 day in Boring 1. Groundwater levels may fluctuate significantly depending on seasonal precipitation and levels of the South Platte River flow. Borings were backfilled with gravel and cement mixture after drilling (with the exception of Borings 1, 7 and 8) and compacted with the weight of the drill rig. Borings 1, 7, and 8 were backfilled with auger cuttings and compacted after drilling. The borings conducted in 6th Avenue were patched with at least 9 inches of Transpatch© High Strength Early set grout that was mixed on site.

# 7.0 LABORATORY TESTING

Laboratory tests conducted on selected soil and bedrock samples consisted of natural moisture contents, dry densities, liquid and plastic limits (Atterberg Limits), grain size distribution (gradation), swell-compression, unconfined compression, R-value, water-soluble sulfate concentrations, and chemical analysis. Laboratory test results are shown on Figures 4 through 24 and summarized on Tables 1 and 2.

**Swell-Compression Tests**: Swell-compression tests are a direct measurement of compressive or expansive potential for a particular sample when wetted. Measurements were made by loading the sample in a consolidometer to a light surcharge pressure, subjecting the sample to wetting, then allowing the specimen to swell or compress. After stabilization, additional loads were applied with each load increment given the opportunity to stabilize. Swell-compression tests were performed in accordance with local practice on samples of the fill consisting of clay and clayey sand and on claystone bedrock.

The results shown on Figures 4 through 7 indicate little or no swell potential under light load and wetting for the samples of soil and bedrock. A low to moderate compressibility under increased loading was also indicated.

**Atterberg Limits and Gradations**: Atterberg limits and gradation analyses were used to classify the soils according to the American Association of State Highway and Transportation Officials (AASHTO) classification system. These classifications provide a qualitative assessment of engineering properties. Gradation analysis and Atterberg Limits test results are presented on Figures 8 through 10.

The Atterberg Limits tests indicate that the man-placed (artificial) fill samples generally had low to high plasticity and underlying natural granular soil samples were mostly non-plastic. An elastic silt with sand was classified for a sample from Boring 6 at 34 feet. Tests on the underlying claystone bedrock samples showed medium plasticity.

The combined gradation and Atterberg Limits indicate that most of the fill samples classified as A-6 with some A-1-a material. The lower natural granular soil samples typically classified as A-1-b in accordance with the AASHTO system.

**R-Value**: Selected bulk samples from the upper embankment fill were tested for R-value, which is an indication of the ability of the soil to transfer traffic loading laterally. Figures 11 through 13 show R-values of 60, 62 and below 5 which indicate relatively high strength (and quality) to very low strength. The low value was from Boring 5 at 1 foot to 5 feet. Based on the test results, highly variable pavement support characteristics for the near surface embankment fill exists.

**Unconfined Compressive Strength:** The unconfined strength is a measurement of compressive strength under axial loading without lateral confinement. The test is useful in evaluating soil or bedrock foundation bearing capacities, and the results are shown on Figures 14 through 24. The values ranged from 3,910 pounds per square foot (psf) to 17,860 psf for the samples of claystone and sandstone bedrock, and 1,750 psf to 3,000 psf for sandy lean clay samples obtained from the embankment fill.

**Water-Soluble Sulfates**: The water-soluble sulfate test is a measurement of the potential degree of sulfate attack on concrete exposed to the onsite soils and bedrock. Sulfate solutions react with tri-calcium aluminate hydrate, which is a normal constituent of Portland Cement concrete, forming calcium sulfo-aluminate hydrate with an accompanying substantial volume expansion which causes cracking. Sulfate expansion problems will typically exist when the soils have concentrations in excess of 0.10%.

The concentrations of water-soluble sulfates measured on selected samples of soil and bedrock ranged from 0.02 to 0.18%. The test results indicate a Class 1 "Severity of Sulfate Exposure" in accordance with Table 601-2 of the Colorado Department of Transportation (CDOT) Standard Specifications for Road and Bridge Construction (2011 Edition). For preliminary design, Class 1 requirements as defined in Section 601.04 Sulfate Resistance should be used for concrete exposed to the near surface soils and bedrock encountered within the project area. During final design, additional sulfate concentration tests should be performed, as needed. Water soluble sulfate test results are summarized in Table 2.

**Other Chemical Tests**: Laboratory test results on selected samples of soil and bedrock indicate electrical resistivity in the range of approximately 170 ohm-cm to 5,000 ohm-cm, pH from 5.5 to 7.6, and chloride concentrations from 0.0015 percent to 0.2181 percent. Sulfides varied from positive to negative detection. Water soluble chloride concentrations and positive or negative sulfide presence were performed by Colorado Analytical Laboratories, Inc., and their results are contained in Appendix A. Remaining chemical tests were performed by Geocal, Inc. Test results are summarized in Table 2.

# 8.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

Two foundation types, driven H-piles and drilled shafts, both supported by the underlying bedrock appear to be suitable for use at this site and for support of the new bridge structure. Driven H-piles will likely encounter refusal within a few feet of the bedrock surface and may be designed for the structural capacity of the piles. Drilled shafts will likely have to be installed using slurry and temporary casing to control ground water, caving, and potentially flowing material. The two foundation types are discussed in the following sections.

## 8.1 Driven Piles

Preliminary recommendations presented in this section are based on the "AASHTO LRFD Bridge Design Specifications" manual, the subsurface data obtained, our experience, and local geotechnical engineering practice. Installation of driven piles should be in accordance with Section 502 "Piling" of the *Standard Specifications for Road and Bridge Construction* (2011 or latest edition), by the Colorado Department of Transportation (CDOT standard specifications).

1. Piles may consist of heavy steel H-sections consisting of Grade A50 steel and driven to refusal in the underlying bedrock. Refusal criteria should be determined during construction using the Pile Driving Analyzer (PDA) in accordance with Section 502 of the CDOT specifications, latest edition.

2. The pile driving contractor should provide the results of a GRLWeap drivability analysis for the pile driving equipment proposed for use, and the type of pile in accordance with the CDOT specifications prior to pile driving operations.

3. Due to the presence of granular soils underlying the embankment, use of a driving shoe may be required to drive the pile through the granular soils and into the underlying bedrock.

4. A combined side shear and end bearing ultimate capacity of 45 kips per square inch (ksi) times the cross sectional area may be used for grade A50 steel for preliminary design. Load and resistance factors used for final design should be consistent with LFRD procedures, as established by AASHTO.

## 8.2 Drilled Shafts

Drilled shafts also appear feasible from a preliminary geotechnical consideration. Casing and slurry installation methods will be required to control caving and ground water. The design and construction criteria presented below should be observed for a drilled shaft foundation system. Installation should be in accordance with Section 503 – Drilled Caissons of the CDOT standard specifications.

1) For preliminary design, drilled shafts may be designed for a nominal tip bearing pressure of 140,000 psf and ultimate side shear value of 14,000 psf for that portion of the foundation in competent bedrock. Load factors used for final design should be consistent with current LFRD procedures as established by AASHTO. A tip resistance factor of 0.55, and a side resistance factor of 0.60 should be applied to the above nominal soil bearing capacity recommendations, as determined by O'Neill and Reese (1999) for drilled shafts in Intermediate Geomaterials (IGMs).

2) The presence of water and caving soils encountered in the exploratory borings indicates that casing and slurry construction methods will be required to reduce water infiltration and caving. If water cannot be removed, or if it is impractical to remove the water prior to placement of concrete, then concrete should be placed using an approved tremie method. The contractor should be advised that water bearing sandstone layers may be encountered.

## 8.3 Lateral Load Capacity

The following preliminary recommendations are based on the structural engineer using the computer program LPILE for the lateral load analysis. We recommend that the bedrock be modeled as hard clay. Lateral capacity parameters are presented below to allow the structural engineer to evaluate possible soil-structure responses under varying conditions and assumptions.

**Preliminary Lateral Capacity Parameters**
**For Drilled Shaft or Driven Pile Foundations**

| Soil Type | Total Unit Weight (pcf) | Cohesion, c (psf) | Friction Angle ($\phi$) | k-static (pci) | $\varepsilon_{50}$ |
|---|---|---|---|---|---|
| Artificial Fill (Embankment Soils) | 125 | 0 | 20 | 70-100 | 0.020 |
| Natural Granular Soils (Submerged) | 65 | 0 | 30 | 125 | --- |
| Bedrock | 125 | 5,000 | 0 | 2,000-3000 | 0.003 |

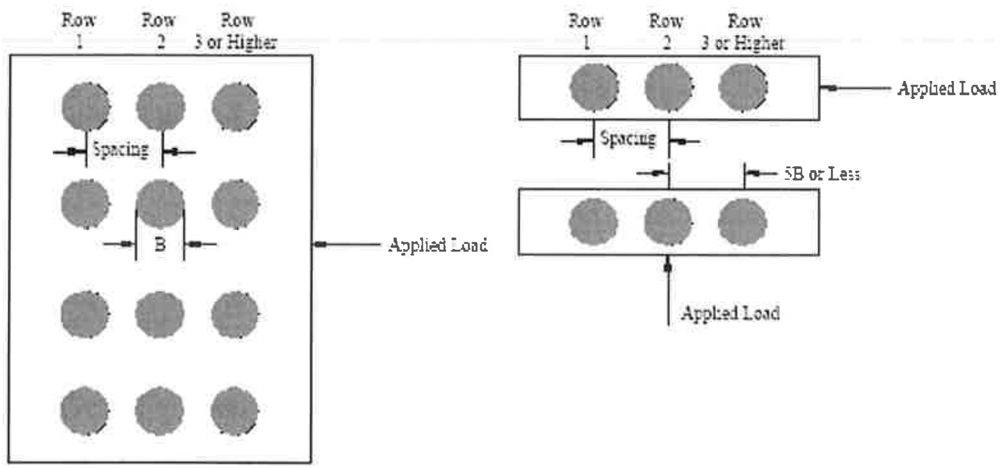
Reductions in lateral capacity for loading perpendicular to the line of shafts or piles will not be required if center to center spacing of 5 shaft or pile diameters or more between adjacent drilled shafts or piles is maintained.

For lateral loads parallel to the line of shafts/piles, reduction in lateral capacity is necessary at a spacing less than 6 diameters. LPILE uses p-multipliers to account for reduced capacity of closely spaced drilled shafts or piles for loading in either direction. Data presented below are from Article 10.7.2.4 of the 2007 AASHTO LRFD Bridge Design Specifications 4th Edition Manual. A sketch of the loading and how the rows are referenced is also shown.

**P-Multipliers**
**Drilled Shaft or Driven Pile Foundation**

| | p-multiplier for LPILE | | |
|---|---|---|---|
| Center to Center Spacing | Row 1 | Row 2 | Row 3 and Higher |
| 3B | 0.7 | 0.5 | 0.35 |
| 4B | 0.85 | 0.67 | 0.52 |
| 5B | 1 | 0.85 | 0.70 |

**B= Diameter of Shaft or Pile**




# 9.0 RETAINING STRUCTURES

The recommendations presented below should be considered preliminary. Additional explorations, analysis, and design recommendations may be required once retaining wall types, locations, and geometries have been established. We have assumed that at least new wing walls may be needed at the bridge abutments.

## 9.1 Gravity and Cantilever Walls

Gravity or cantilevered retaining walls should be supported by the same foundation type as the bridge foundations (driven piles or drilled shafts). Retaining structures that are laterally supported and can be expected to undergo only a slight amount of deflection should be designed for lateral earth pressures based on the "at-rest" earth pressure condition. Cantilevered or gravity retaining structures which rotate and/or deflect sufficiently to mobilize the internal soil strength of the wall backfill may be designed for the "active" earth pressure condition. The following ultimate earth pressure coefficients may be used for imported Class 1 material to be used as backfill.

| Material | Active ($K_a$) | At-Rest ($K_o$) | Passive ($K_p$) | $\gamma_T$ – Unit Weight (pcf) | Friction Angle ($\phi$), degrees |
|---|---|---|---|---|---|
| Imported Class 1 | 0.28 | 0.44 | 3.54 | 130 | 34 |

Lateral wall movements or rotation of at least 0.1% of the wall height is typically required to develop the full active case, whereas lateral movement of at least 2% of the wall height is normally required to establish the full passive case assuming granular Class 1 backfill. Suitable factors of safety should therefore be applied to the above ultimate values to limit strain needed to reach ultimate strength, particularly with passive resistance where large strains are needed to mobilize full resistance. Imported material should meet CDOT Class 1 structure backfill grading requirements. Equivalent fluid unit weights should be taken as follows:

| | | | |
|---|---|---|---|
| Above ground water: | $\gamma_{eq}$ | = | $\gamma_T \times K_{a,o,p}$ |
| Below ground water: | $\gamma_{eq}$ | = | $(\gamma_T - 62.4) \times K_{a,o,p}$ |
| where | $\gamma_T$ | = | soil total unit weight |
| | $K_{a,o,p}$ | = | appropriate earth pressure coefficient |

The above parameters are for a horizontal backfill and no surcharge loading. Foundation and retaining structures should be designed for appropriate surcharge pressures such as from traffic, etc. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on retaining structures. An under-drain should be provided to help prevent hydrostatic pressure buildup, unless the wall is designed to accommodate the additional pressure.

Care should be taken not to over-compact the backfill or use large equipment adjacent to the wall because this could cause excessive lateral wall loading.

### 9.2 Soil Nail Walls

Soil nail walls installed in the embankment fill appear to be feasible from a geotechnical consideration. Most of the soil encountered in the embankment areas were granular with variable amounts of clay and silt. Final design of soil nail walls should be developed using parameters determined with site specific subsurface investigations and appropriate laboratory analyses. Procedures developed by the Federal Highway Administration should be used for design, construction and testing.

For feasibility or preliminary evaluations of soil nail walls, the length of the nails may be assumed to be about 70% of the wall height. The following additional parameters may be assumed for preliminary design.

| | |
|---|---|
| Total Unit weight (pcf), $\gamma_T$ | 125 |
| Friction angle (degrees), $\phi$ | 28 |
| Cohesion (psf), C | 200 |

## 10.0 SEISMIC DESIGN PARAMETERS

The structure is located at Latitude 37.725746 and Longitude 105.010884 within the South Platte River Terrace Deposits. The borings conducted by Geocal indicate that the underlying soils generally consisted of relatively deep (35 feet to 39 feet) man-placed artificial fill (silty to clayey sand with gravel) underlain by natural soils (sand and gravel with some clay and silt) to depths of 48 feet to 53 feet. Very hard claystone bedrock was encountered below the natural soils and extended to the maximum depth explored, 85 feet. We have assumed that the bridge will be supported by either drilled shafts or driven piles extended into the underlying claystone bedrock. Based on the amount of overburden soils present, the Seismic Site Class utilized for analysis should be Site Class D (stiff soils). There is low potential for liquefaction of the soils encountered below the groundwater table in a seismic event; however, any potential liquefaction should have limited effect on the bridge structure's foundations because the foundations will be supported by bedrock and the depth of potentially liquefiable material is nominal.

The nearest potentially active fault identified based on United States Geological Survey (USGS) is the Ute Pass Fault zone which is located approximately 32 miles to the southwest. Other seismic hazards such as ground rupture or faulting and slope instability have low risk of occurrence at the site. The bridge should be deemed a critical structure based on the expected usage.

Utilizing the AASHTO Specifications for LRFD Seismic Bridge Design, the site is classified as "D" and the seismic zone as "1" using Tables 3.10.3.1-1 and 3.10.6-1, as shown in the LRFD design guidelines. Using the AASHTO Earthquake Motion Parameters program, the seismic design spectrum plots were created for Spectral Acceleration vs. Time and Spectral Acceleration vs. Spectral Displacement for the site Class B and Site Class D responses. We have attached the printouts of the graphs and data generated from the AASHTO program in Appendix B. For preliminary design, the following parameters may be utilized for design of the bridge structure:

- Peak Ground Acceleration (PGA): 0.059 – Site Class B
- Spectral Acceleration Coefficient at 0.2 Seconds ($S_s$): 0.126 – Site Class B
- Spectral Acceleration Coefficient at 1.0 Seconds ($S_1$): 0.034 – Site Class B
- Modified Peak Ground Acceleration ($A_S$): 0.095 – Site Class D
- Modified Spectral Acceleration Coefficient at 0.2 Seconds ($SD_s$): 0.202 – Site Class D
- Modified Spectral Acceleration Coefficient at 1.0 Seconds ($SD_1$): 0.081 – Site Class D

# 11.0 UNDERDRAIN SYSTEM

Below grade structures should be provided with an underdrain system which will help prevent buildup of hydrostatic pressures. The underdrain system should consist of a perforated PVC pipe surrounded by free draining granular material placed at the bottom of the wall backfill and sloped at a minimum 1% grade to a suitable gravity outlet. Free draining granular material used in the drain system should conform to the requirements for Class B filter material as specified in the CDOT standard specifications.

# 12.0 SITE GRADING

Based on the materials encountered, excavation of the onsite materials should be possible with conventional heavy duty excavating equipment. Most of the embankment material is expected to be granular (sand and gravel) with mixed clays and silts. The natural soils below the embankment fill are expected to be mostly sands and gravels. Site grading activity should be conducted in accordance with the Colorado Department of Transportation *Standard Specifications for Road and Bridge Construction* (latest edition).

The re-use of onsite materials will be a function of what the intended use is. Most of the material is expected to be granular, although some clays and organic material should be anticipated. Clays and organic material should be kept outside of areas planned for pavements, structure backfill, or use as fill for support of structures. Soils used for support of pavements should meet the minimum strength requirements as specified during final design.

Permanent un-retained cuts in the overburden soils up to 10 feet high should be no steeper than 3:1 horizontal to vertical grade unless evaluated individually. The risk of slope instability will be significantly increased if seepage is encountered in cuts. If seepage on slopes is encountered, stability should be evaluated. Good surface drainage should be provided around permanent cuts to direct surface runoff away from the cut face. Cut slopes and other stripped areas should be protected against erosion by vegetation or other methods.

Fill slopes should be constructed no steeper than 3:1 horizontal to vertical grade provided the fills are properly compacted and drained. The ground surface underlying proposed fills should be carefully prepared by removing organic matter, scarifying to a depth of 12 inches and re-compacting in accordance with the CDOT standard specification. Fills should be benched into hillsides that are steeper than 4 horizontal to 1 vertical. Settlement of embankments constructed of granular material similar to that encountered onsite and properly compacted, should be less than 1% of the embankment height and essentially occur during construction.

If sloped excavations are used, stockpiled material should be placed no closer than 10 feet to the top of the excavation. Sloped and braced excavations should conform to applicable OSHA regulations, and the contractor should assume responsibility for an excavation that is safe for workers.

# 13.0 PRELIMINARY PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade without overstressing the subgrade soils. Performance of the pavement structure is a function of a number of factors including but not limited to the physical properties of the subgrade soils, drainage, climate, and traffic loading. The preliminary pavement sections presented in this section are based on laboratory test results and CDOT and AASHTO design procedures, and apply to the 6th Avenue approaches to the bridge.

**Traffic Loading and ESAL Calculations:** The CDOT web site was used to obtain Annual Average Daily Traffic Volumes for 6th Avenue from near the intersection of Sheridan Boulevard. These volumes were then utilized to determine the 20 year 18 kip Equivalent Single Axle Loadings ($ESAL_{20}$) for asphalt pavement, and the 30 year $ESAL_{30}$ for concrete pavements. The CDOT website presents the route and reference points (mile posts) and provides the traffic data for those points and when data was gathered. The data is displayed as annual average daily traffic (AADT) with breakdown of single unit trucks and combination unit trucks. Information from the website is included in Appendix B.

For preliminary design, we assumed a 3% annual traffic growth rate. Traffic volumes were projected 20 years and 30 years based on the 2010 traffic data. Based on the CDOT website, vehicle distributions were 96.5% passenger vehicles, 2.4% single unit trucks, and 1.1% combination unit trucks. A design lane factor of 30% was used to distribute the total traffic across the 6 lanes, (3 lanes in each direction).

An 18 kip Equivalent Single Axle Load (ESAL) is the equivalent 18,000 pound axle loading for the different vehicle types. The design ESAL is the total number of equivalent loadings to either asphalt or concrete pavements over the design period. The design ESALs presented here should be considered preliminary and will need to be checked during final design with more specific and current traffic data. The following values were calculated (included in Appendix B): $ESAL_{20}$ = 8,284,671 for asphalt (HMAP) and $ESAL_{30}$ = 13,769,951 for concrete (PCCP).

**General Design Parameters:** The following summarizes the pavement design parameters used:

| | |
|---|---|
| General | |
| Initial Serviceability | 4.5 |
| Terminal Serviceability | 2.5 |
| Reliability | 95% |
| Drainage Coefficient | 1.0 |
| Growth rate | 3.0% |
| Concrete | |
| Overall Standard Deviation | 0.34 |
| Loss of Support | 1.0 |
| Modulus of Rupture | 650 psi |
| Concrete Modulus of Elasticity | 3.4 million psi |
| Load Transfer Coefficient (doweled and tied) | 2.8 |
| $ESAL_{30}$ | 13,769,951 |
| Asphalt | |
| Structural Coefficient (HMAP) | 0.44 |
| Structural Coefficient (ABC) | 0.12 |
| $ESAL_{20}$ | 8,284,671 |

**Subgrade Soil Strength Coefficients:** The pavement subgrade soils encountered classified between A-1-a and A-6 in accordance with the AASHTO classification system. Laboratory R-values measured from 62 to less than 5, indicating a high variability for the pavement subgrade soil in the approach areas. Good to very poor subgrade support characteristics could be exposed during construction. For design purposes, we assigned an R-value of 50, indicating that any poor subgrade (R-value less than 50) encountered within the pavement areas will need to be subexcavated a minimum of 3 feet and replaced with R-50 or better material.

A resilient modulus of 13,168 was determined based on the CDOT equations 2.1 and 2.2 in the 2012 CDOT Pavement Design Manual. For rigid pavement thickness calculations, a k-value (modulus of vertical subgrade reaction) of 175 pounds per cubic inch (pci) was chosen based on Table 2.3 of the design manual. The strength values used for pavement design are summarized as follows:

| R-Value | Resilient Modulus (psi) | k-value (pci) |
|---|---|---|
| 50 | 13,168 | 175 |

**Pavement Thickness Recommendations:** Hot Mix Asphalt Pavement (HMAP) thickness sections were calculated using AASHTOWare DARWin software, following CDOT and AASHTO guidelines. Portland Cement Concrete Pavement (PCCP) thickness sections were calculated using the AASHTO 1998 Rigid Pavement Design Guide software provided by the FHWA.

The recommended pavement thickness sections shown below are for R-value 50 material in the upper 3 feet of the subgrade, 20 year design for asphalt pavement (HMAP), 20 year design for asphalt over aggregate base course (ABC) pavement, and 30 year design for concrete pavement (PCCP). Design printouts are included in Appendix B.

| Full depth HMAP (inches) | HMAP over ABC (inches) | PCCP over ABC (inches) |
|---|---|---|
| 9 ½ | 7 ½ over 6 | 11 ¼ over 6 |

A 6 inch layer of aggregate base course should be used to support concrete pavements. This ABC layer will help control the effect of fines migration through the joints and subsequent loss of support.

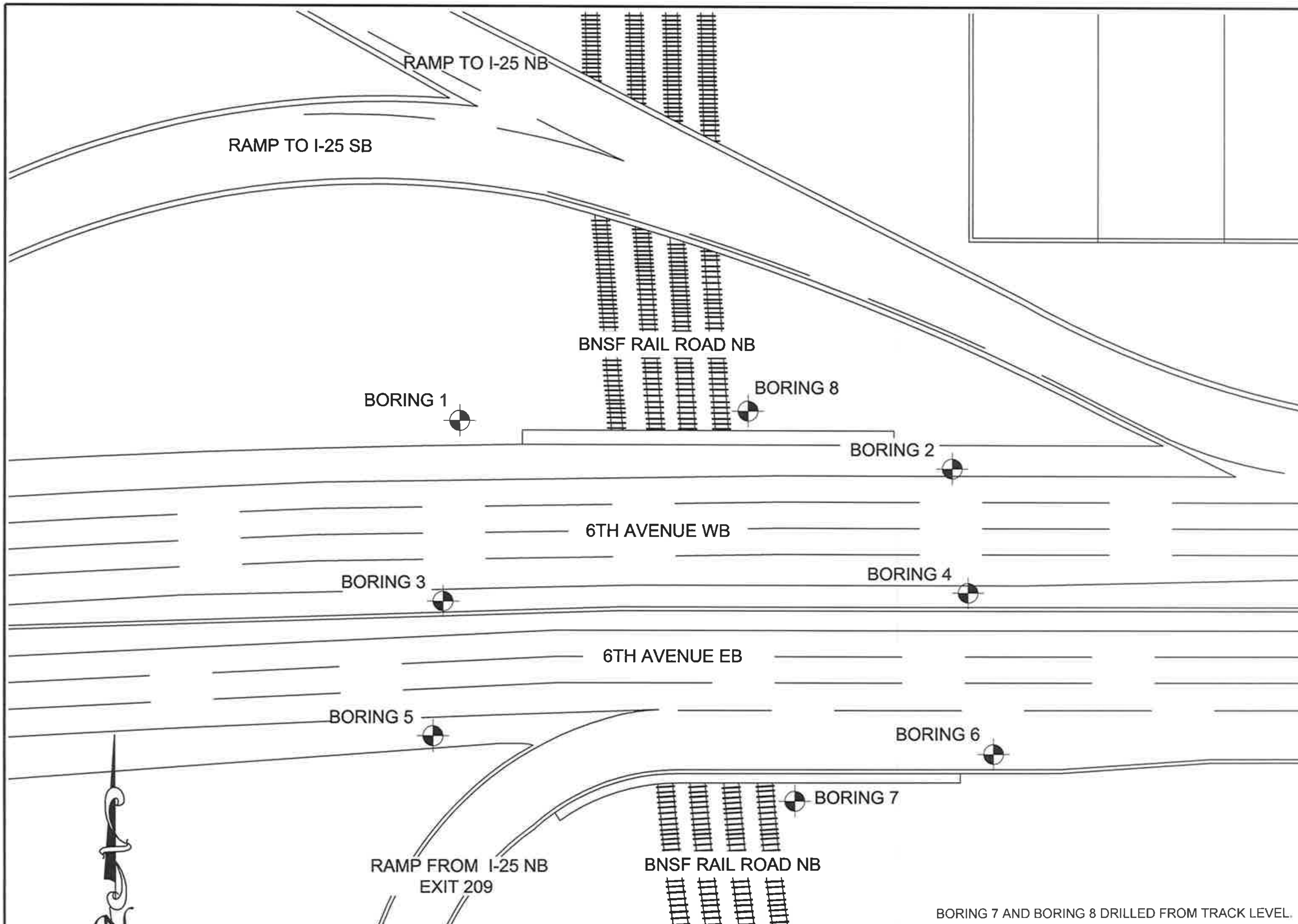
During final design the HMAP, PCCP and ABC pavement sections should checked with more current data. The design should meet the requirements of the Colorado Department of Transportation (CDOT) and/or the standards of the Metropolitan Government Pavement Engineers Council (MGPEC) as applicable.

# 14.0 LIMITATIONS

This 30% design level report has been prepared in accordance with generally accepted geotechnical engineering practices in this area, and is provided for use by the client for preliminary design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings drilled at the approximate locations shown on Figure 1. Additional explorations for the structures, walls, and pavements are recommended for final design. The nature and extent of variations between the borings may not become evident until excavation is performed.

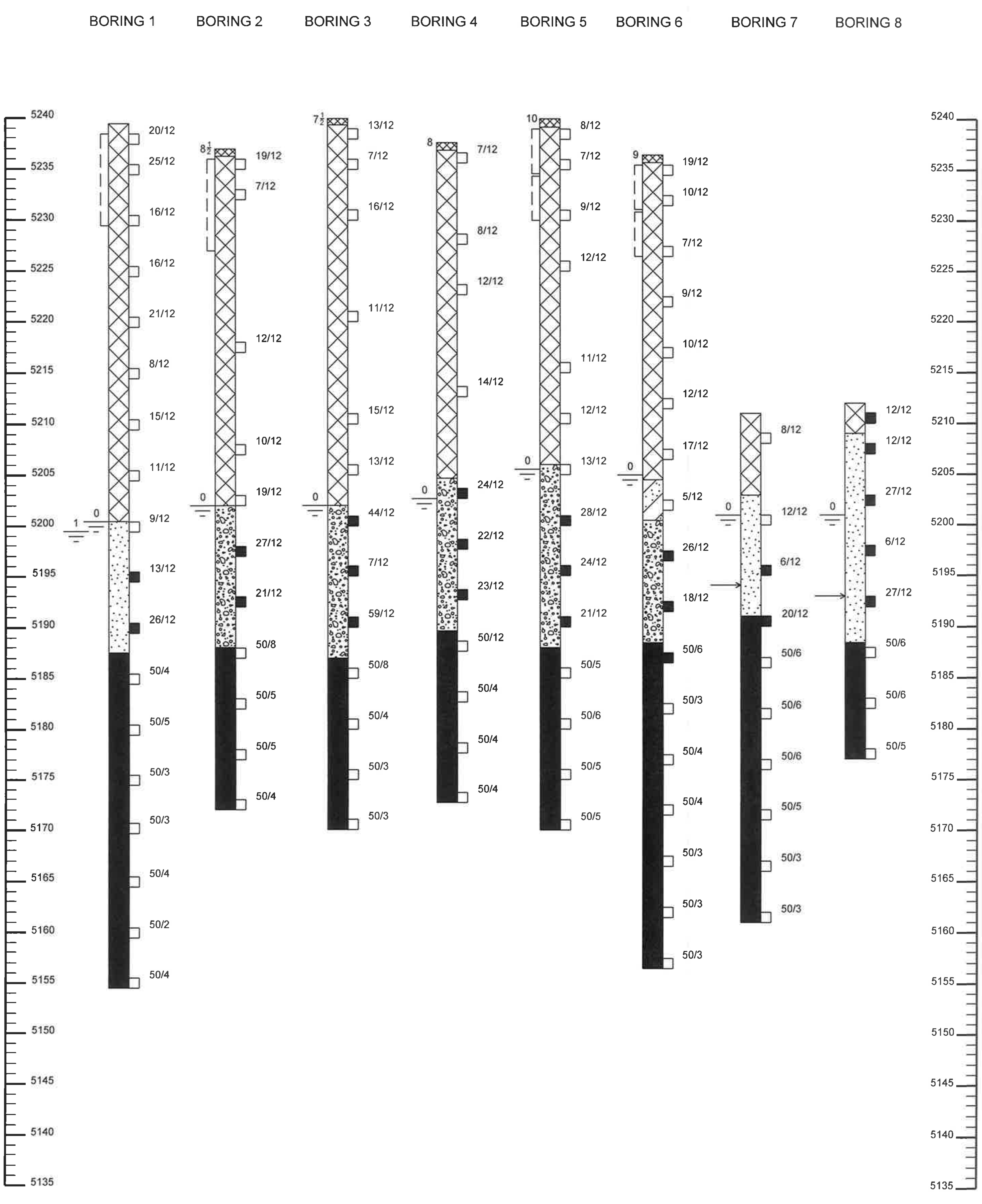
Geocal's professional services were performed using the degree of care and skill ordinarily exercised by reputable geotechnical engineers practicing in this or similar environments. No warranty expressed or implied is made. Geocal is not responsible for the interpretation of the site surface and subsurface conditions by others that are not consistent with the contents of this report.

Investigations into the occurrence or potential occurrence of hazardous materials, or other environmental assessments that may be applicable to the site are beyond the scope of services represented by this report. On-site observation of excavations and foundation bearing strata and testing of geotechnical materials by a representative of this office is recommended.




BORING 7 AND BORING 8 DRILLED FROM TRACK LEVEL.

| | | | |
|---|---|---|---|
| G10.1354.002 | GEOCAL, INC. | 6TH AVENUE BRIDGE OVER BNSF<br>LOCATIONS OF EXPLORATORY BORINGS | FIGURE 1 |




| G10.1354.002 | GEOCAL, INC. | 6TH AVENUE OVER BNSF<br>LOGS OF EXPLORATORY BORINGS | FIGURE 2 |
|---|---|---|---|

## LEGEND

$8\frac{1}{2}$ ASPHALT, approximate thickness in inches indicated in the top left corner.

FILL, gravel and sand with silt, trace clay, occassionally cobbley, medium dense with some loose zones, moist, fine to coarse grained sand, small to large gravel, light to dark brown, some asphalt, glass, and construction debris.

SAND with GRAVEL, medium dense, medium to coarse grained sand, small gravel, wet, brown.

SAND and SILT, dense, fine grained, low plasticity, wet, black to dark brown, some organic material.

GRAVEL, medium dense to very dense, small to medium gravel, rounded to sub-rounded, wet, brown to dark brown, some clay and sand seams.

CLAYSTONE BEDROCK, mostly with slight sand and slight silt to silty, very hard, slightly moist, blue to dark gray, very fine to fine grained sand.

20/12 Drive sample blow count, Indicates that 20 blows from a 140 pound hammer falling 30 inches were required to drive the California or SPT sampler 12 inches.

2 inch I.D. California liner drive sample.

Standard Penetration Test, $1\frac{3}{8}$ inch I.D., split spoon drive sample.

0,1 Indicates depth to water level and number of days after drilling measurement was made.

Indicates depth to which caved material accumulated.

Indicates disturbed bulk sample.

## NOTES

1. Borings 1 through 6 were drilled on October 31 to November 9, 2011 with CM-75 drill rig equipped with 3 $\frac{1}{4}$ inch inside diameter hollow-stem augers. Borings 7 and 8 were drilled February 24, 2012 with a CME-550 drill rig equipped with $3\frac{1}{4}$ inch inside diameter hollow-stem augers.

2. Location of borings shown on Figure 1 are approximate.

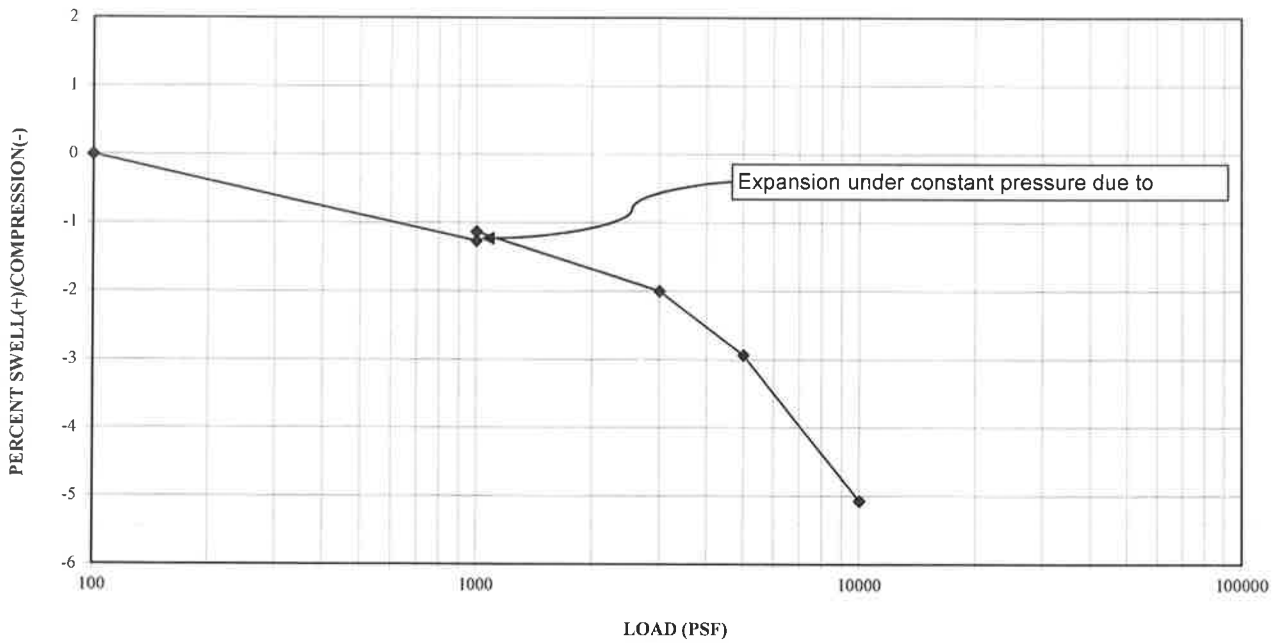
3. The lines between strata represent approximate boundaries between material types. Transitions between materials may actually be gradual.

4. Boring logs are drawn to elevation.

5. Water level readings shown on the logs were made at the time and under conditions indicated, fluctuations in the water level may occur with time.

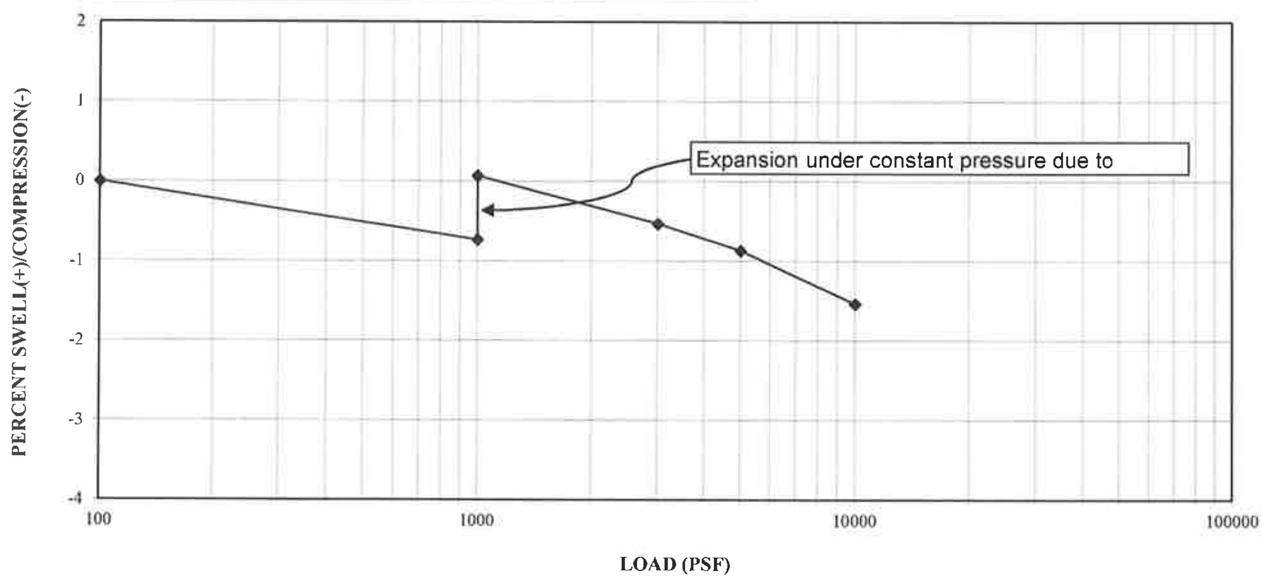
| | | | |
|---|---|---|---|
| G10.1354.002 | GEOCAL, INC. | 6TH AVENUE OVER BNSF<br>LEGEND AND NOTES FOR EXPLORATORY BORINGS | FIGURE 3 |

## SWELL-COMPRESSION TEST




| | |
|---|---|
| **Sample Location** | Boring 1 |
| **Sample Depth** | 24 feet |
| **Sample Description** | Sandy lean clay, fill |
| **USCS Classification** | CL |
| **AASHTO Classification** | |

| | |
|---|---|
| **Dry Density** | 107 pcf |
| **Moisture Content** | 20.7 % |
| **Volume Change** | 0.1 % |
| **Swell Pressure** | 0 psf |

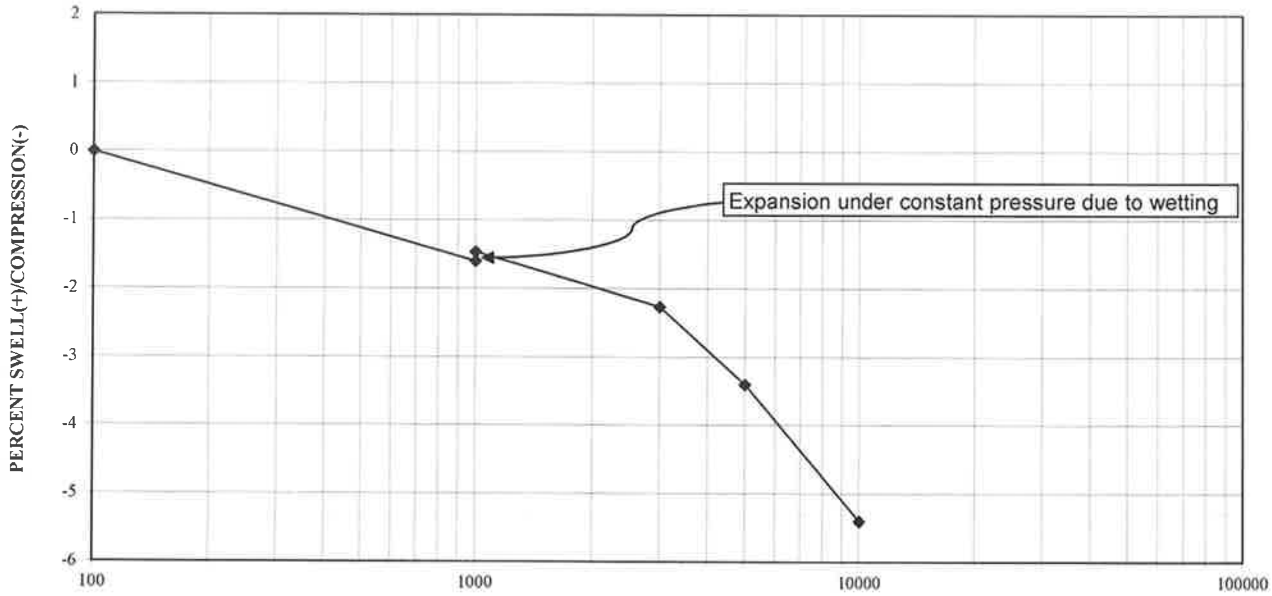



| | |
|---|---|
| **Sample Location** | Boring 1 |
| **Sample Depth** | 59 feet |
| **Sample Description** | Claystone bedrock |
| **USCS Classification** | |
| **AASHTO Classification** | |

| | |
|---|---|
| **Dry Density** | 112 pcf |
| **Moisture Content** | 17.3 % |
| **Volume Change** | 0.8 % |
| **Swell Pressure** | 1,030 psf |

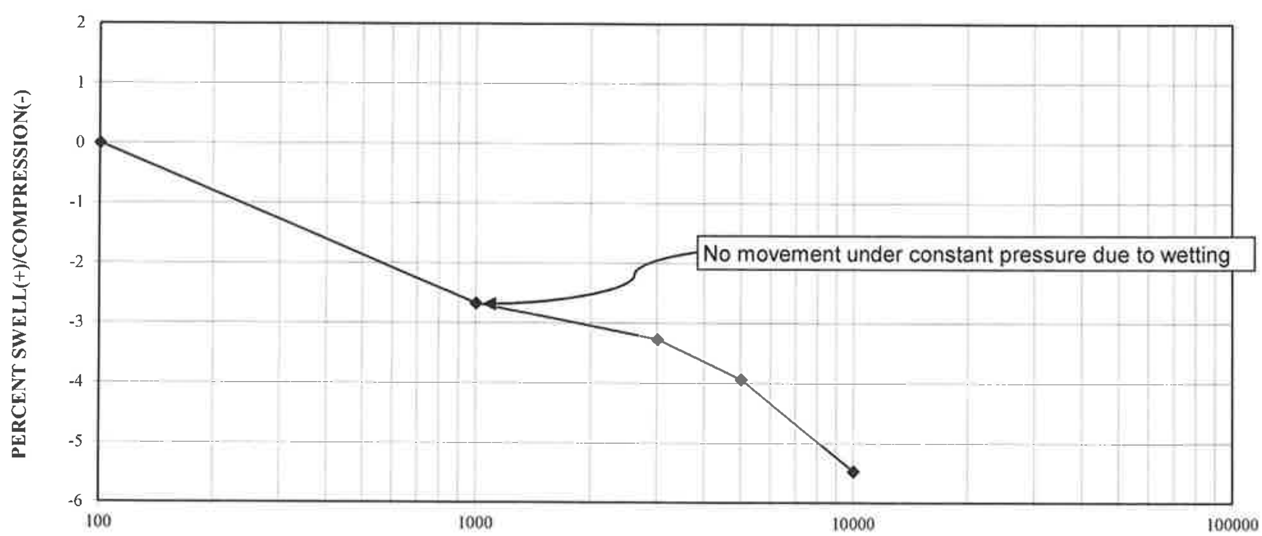
| | | |
|---|---|---|
| GEOCAL, INC. | **6th Avenue over BNSF** | **JOB NO.** G10.1354.002 |
| | **SWELL - COMPRESSION TEST RESULTS** | **FIGURE NO.** 4 |

## SWELL-COMPRESSION TEST




| | |
|---|---|
| **Sample Location** | Boring 3 |
| **Sample Depth** | 34 feet |
| **Sample Description** | Fat clay with sand, fill |
| **USCS Classification** | CH |
| **AASHTO Classification** | A-7-6(25) |

| | |
|---|---|
| **Dry Density** | 92 pcf |
| **Moisture Content** | 32.9 % |
| **Volume Change** | 0.1 % |
| **Swell Pressure** | 0 psf |

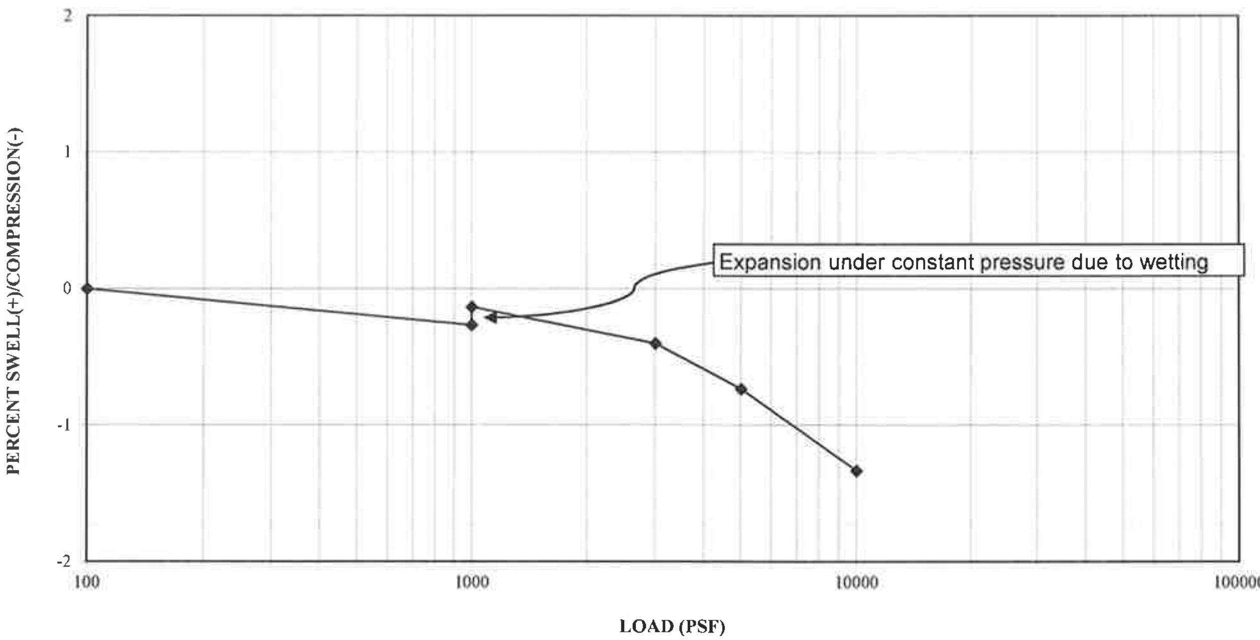



| | |
|---|---|
| **Sample Location** | Boring 4 |
| **Sample Depth** | 24 feet |
| **Sample Description** | Clayey sand with gravel, fill |
| **USCS Classification** | SC |
| **AASHTO Classification** | |

| | |
|---|---|
| **Dry Density** | 110 pcf |
| **Moisture Content** | 17.4 % |
| **Volume Change** | 0.0 % |
| **Swell Pressure** | 0 psf |

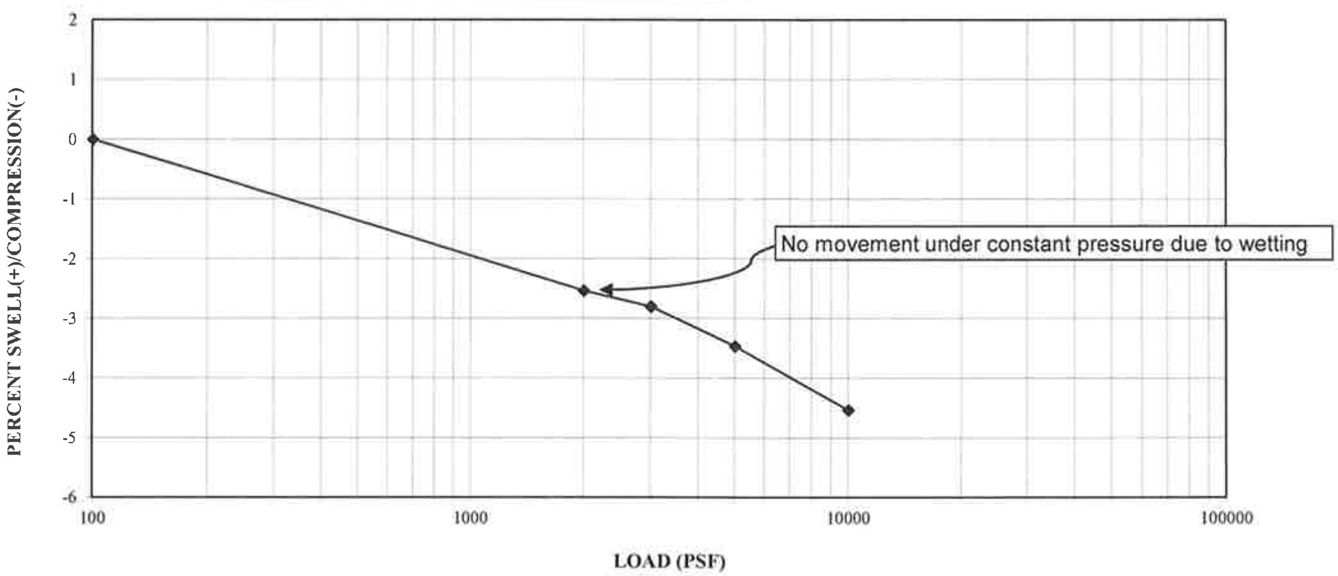
| | | |
|---|---|---|
| **GEOCAL, INC.** | **6th Avenue over BNSF** | **JOB NO.** G10.1354.002 |
| | **SWELL - COMPRESSION TEST RESULTS** | **FIGURE NO.** 5 |

## SWELL-COMPRESSION TEST




| | |
|---|---|
| **Sample Location** | Boring 4 |
| **Sample Depth** | 54 feet |
| **Sample Description** | Sandstone bedrock |
| **USCS Classification** | |
| **AASHTO Classification** | |

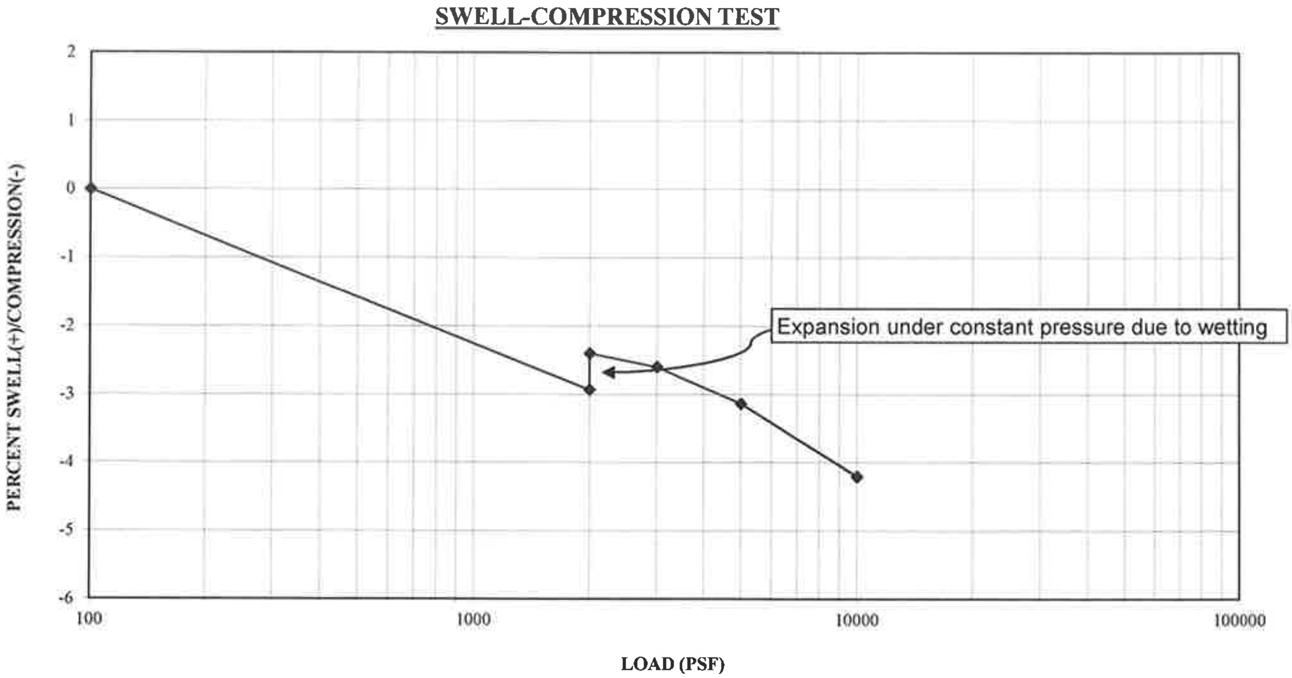
| | |
|---|---|
| **Dry Density** | 112 pcf |
| **Moisture Content** | 16.3 % |
| **Volume Change** | 0.1 % |
| **Swell Pressure** | 0 psf |




| | |
|---|---|
| **Sample Location** | Boring 5 |
| **Sample Depth** | 54 feet |
| **Sample Description** | Claystone bedrock |
| **USCS Classification** | |
| **AASHTO Classification** | |

| | |
|---|---|
| **Dry Density** | 110 pcf |
| **Moisture Content** | 16.0 % |
| **Volume Change** | 0.0 % |
| **Swell Pressure** | 0 psf |

| | | | |
|---|---|---|---|
| **GEOCAL, INC.** | **6th Avenue over BNSF** | **JOB NO.** | **G10.1354.002** |
| | **SWELL - COMPRESSION TEST RESULTS** | **FIGURE NO.** | **6** |

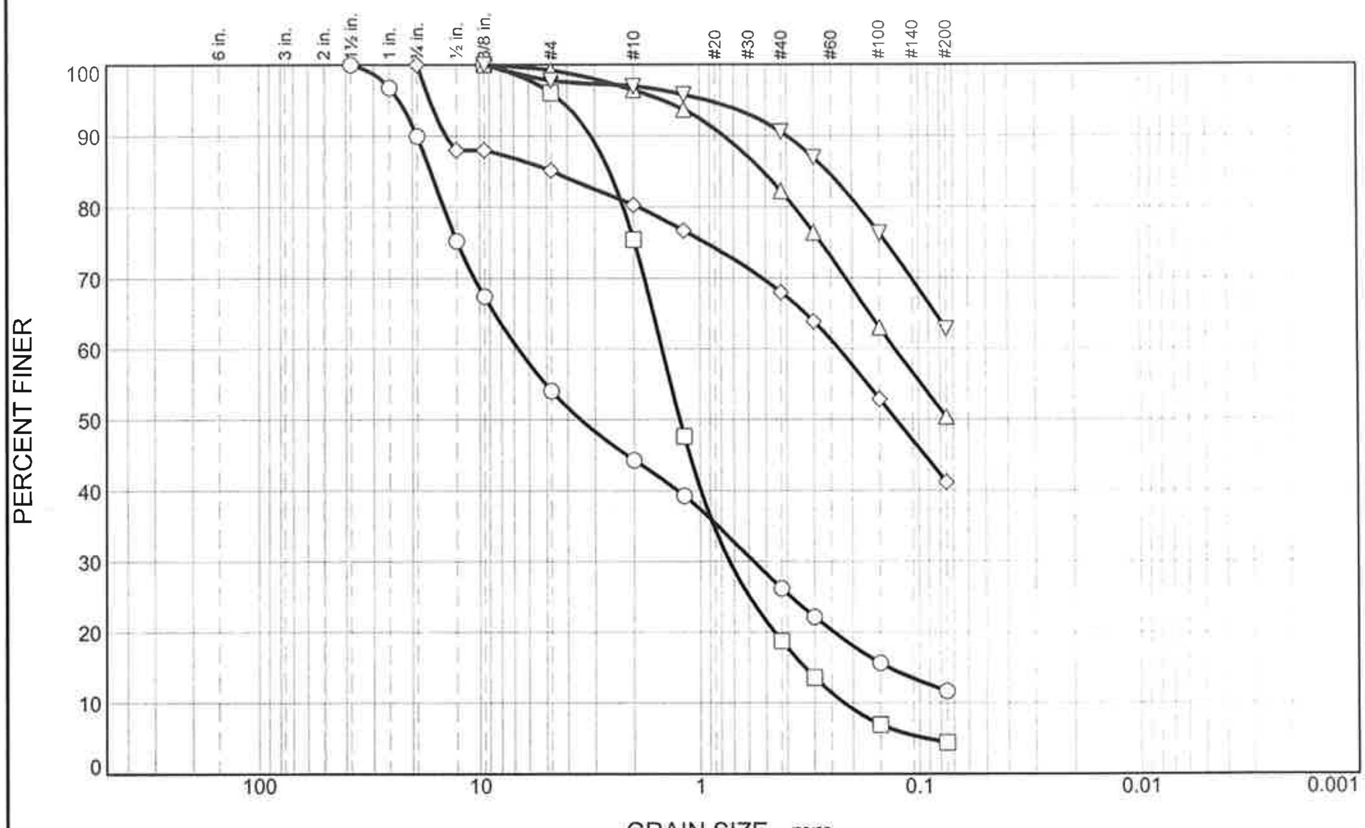



| | |
|---|---|
| **Sample Location** | Boring 6 |
| **Sample Depth** | 49 feet |
| **Sample Description** | Claystone bedrock |
| **USCS Classification** | |
| **AASHTO Classification** | |

| | |
|---|---|
| **Dry Density** | 110 pcf |
| **Moisture Content** | 15.6 % |
| **Volume Change** | 0.5 % |
| **Swell Pressure** | 0 psf |

| | | |
|---|---|---|
| **GEOCAL, INC.** | **6th Avenue over BNSF** | **JOB NO.** G10.1354.002 |
| | **SWELL - COMPRESSION TEST RESULTS** | **FIGURE NO.** 7 |

# Gradation Test Results




| | % +3" | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|
| ○ | 0 | 46 | 42 | 12 | |
| □ | 0 | 4 | 92 | 4 | |
| △ | 0 | 1 | 49 | 50 | |
| ◇ | 0 | 15 | 44 | 41 | |
| ▽ | 0 | 2 | 35 | 63 | |

| | LL | PL | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ○ | 21 | 15 | 16.5413 | 6.7271 | 3.5052 | 0.5704 | 0.1380 | | | |
| □ | NV | NP | 2.5776 | 1.4893 | 1.2374 | 0.7341 | 0.3329 | 0.2202 | 1.64 | 6.76 |
| △ | 37 | 16 | 0.5136 | 0.1281 | | | | | | |
| ◇ | 37 | 15 | 4.6349 | 0.2307 | 0.1268 | | | | | |
| ▽ | 44 | 18 | 0.2588 | | | | | | | |

| Material Description | USCS | AASHTO |
|---|---|---|
| ○ poorly graded gravel with silty clay and sand, fill | GP-GC | A-1-a |
| □ well-graded sand | SW | A-1-b |
| △ sandy lean clay, fill | CL | A-6(7) |
| ◇ clayey sand with gravel, fill | SC | A-6(4) |
| ▽ sandy lean clay, fill | CL | A-7-6(14) |

**Project No.** G10.1354.002 **Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

○ **Location:** Boring 1 **Depth:** 1-10 feet **Sample Number:** 5815-1

□ **Location:** Boring 1 **Depth:** 49 feet **Sample Number:** 5815-4

△ **Location:** Boring 2 **Depth:** 4 feet **Sample Number:** 5830-1

◇ **Location:** Boring 2 **Depth:** 34 feet **Sample Number:** 5830-2

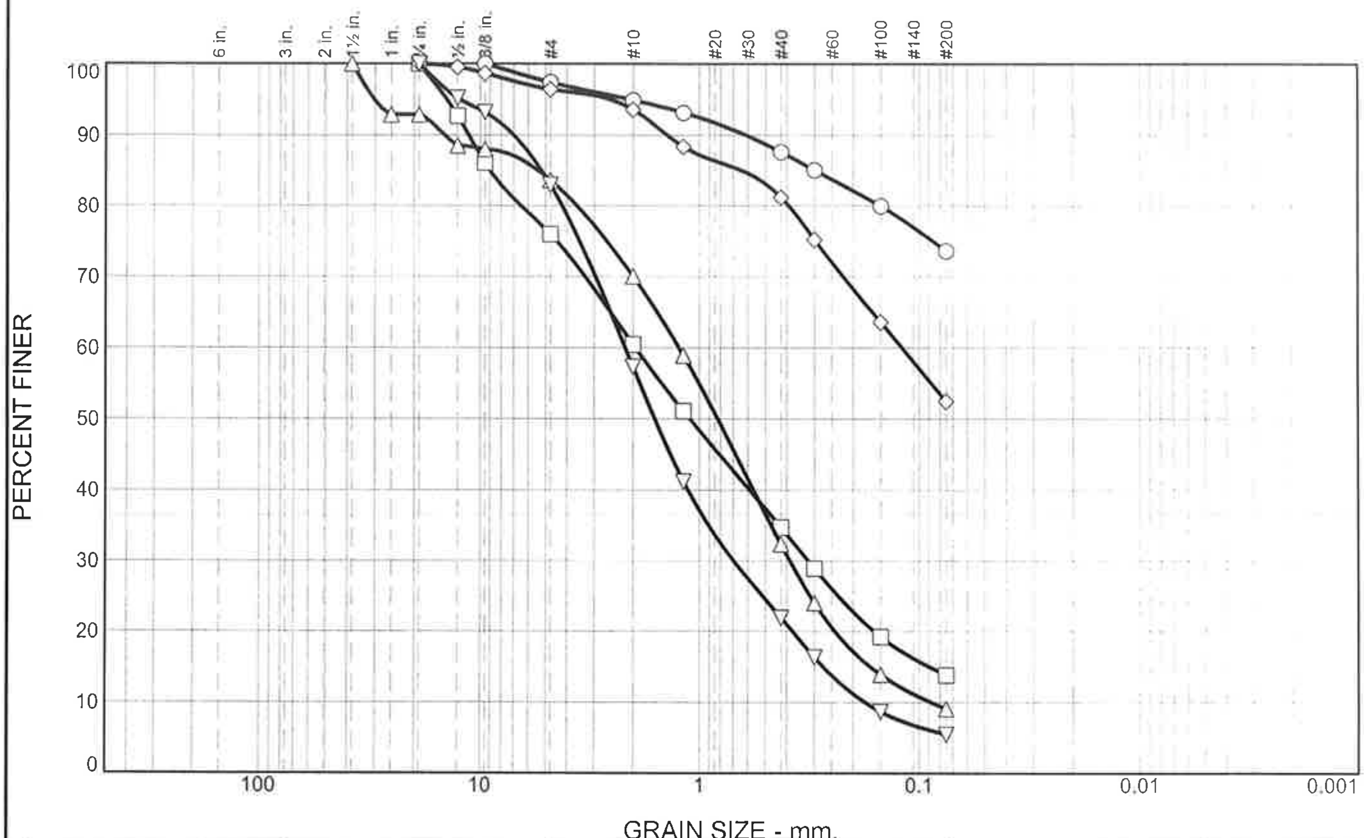
▽ **Location:** Boring 3 **Depth:** 4 feet **Sample Number:** 5820-1

**Remarks:**

# GEOCAL, INC.

**Figure** 8

# Gradation Test Results




| | % +3" | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|
| ○ | 0 | 3 | 23 | 74 | |
| □ | 0 | 24 | 62 | 14 | |
| △ | 0 | 16 | 75 | 9 | |
| ◇ | 0 | 4 | 44 | 52 | |
| ▽ | 0 | 17 | 78 | 5 | |

| | LL | PL | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ○ | 57 | 23 | 0.2980 | | | | | | | |
| □ | NV | NP | 9.0089 | 1.9535 | 1.1056 | 0.3205 | 0.0898 | | | |
| △ | NV | NP | 5.3989 | 1.2371 | 0.8253 | 0.3876 | 0.1670 | 0.0879 | 1.38 | 14.08 |
| ◇ | 40 | 16 | 0.6410 | 0.1200 | | | | | | |
| ▽ | NV | NP | 5.2220 | 2.1705 | 1.5958 | 0.6967 | 0.2735 | 0.1762 | 1.27 | 12.32 |

| Material Description | USCS | AASHTO |
|---|---|---|
| ○ fat clay with sand, fill | CH | A-7-6(25) |
| □ silty sand with gravel | SM | A-1-b |
| △ well-graded sand with silt and gravel | SW-SM | A-1-b |
| ◇ sandy lean clay, fill | CL | A-6(9) |
| ▽ well-graded sand with silt and gravel | SW-SM | A-1-b |

**Project No.** G10.1354.002 **Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

○ **Location:** Boring 3 **Depth:** 34 feet **Sample Number:** 5820-3

□ **Location:** Boring 3 **Depth:** 44 feet **Sample Number:** 5820-4

△ **Location:** Boring 4 **Depth:** 39 feet **Sample Number:** 5820-8

◇ **Location:** Boring 5 **Depth:** 1-5 feet **Sample Number:** 5830-4

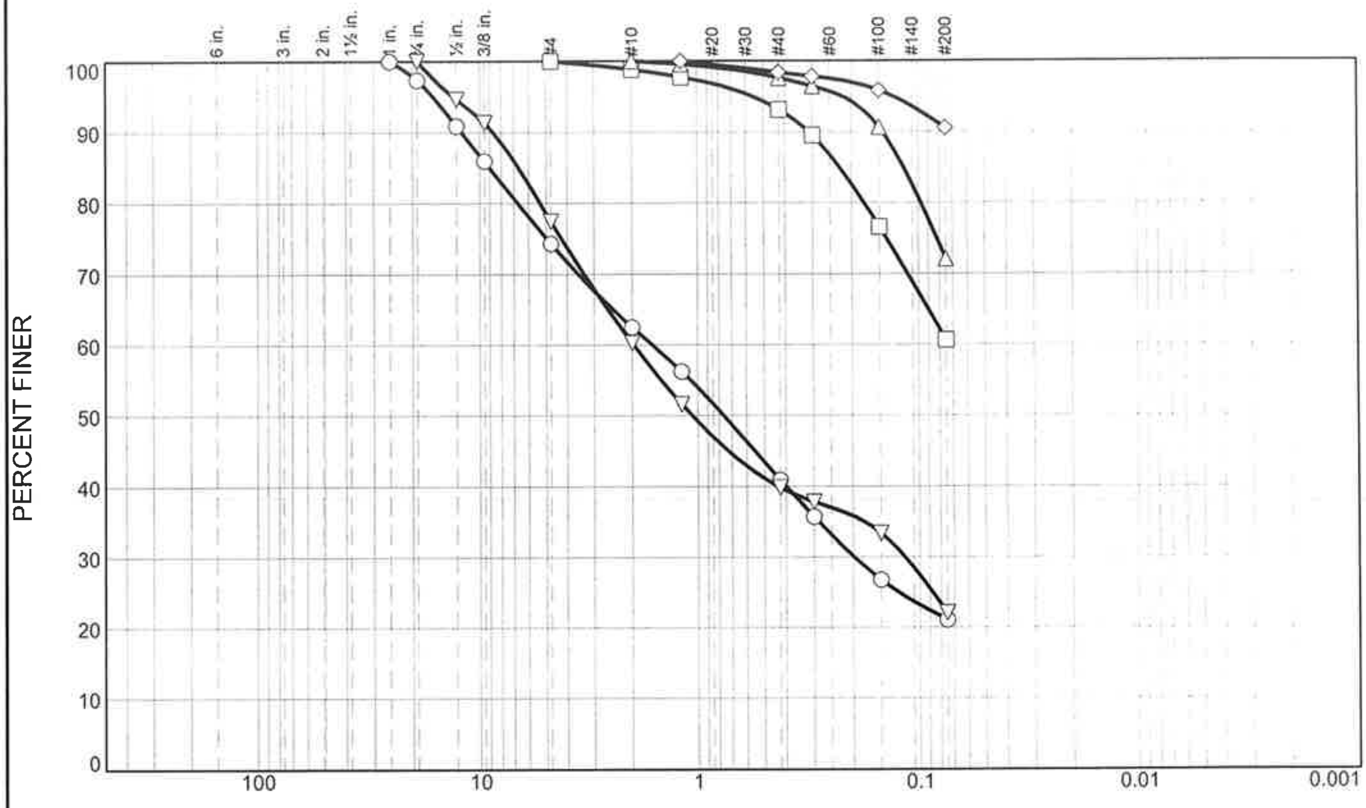
▽ **Location:** Boring 5 **Depth:** 44 feet **Sample Number:** 5830-5

**Remarks:**

# GEOCAL, INC.

**Figure** 9

# Gradation Test Results




| | % +3" | % Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|
| ○ | 0 | 26 | 53 | 21 | |
| □ | 0 | 0 | 39 | 61 | |
| △ | 0 | 0 | 28 | 72 | |
| ◇ | 0 | 0 | 9 | 91 | |
| ▽ | 0 | 23 | 55 | 22 | |

| | LL | PL | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ○ | 26 | 16 | 9.0144 | 1.6148 | 0.7630 | 0.1975 | | | | |
| □ | 41 | 16 | 0.2258 | | | | | | | |
| △ | 52 | 31 | 0.1159 | | | | | | | |
| ◇ | 43 | 23 | | | | | | | | |
| ▽ | 23 | 12 | 6.6759 | 1.9694 | 1.0602 | 0.1167 | | | | |

| Material Description | USCS | AASHTO |
|---|---|---|
| ○ clayey sand with gravel, fill | SC | A-2-4(0) |
| □ sandy lean clay, fill | CL | A-7-6(12) |
| △ elastic silt with sand | MH | A-7-5(16) |
| ◇ claystone bedrock | CL | A-7-6(20) |
| ▽ clayey sand with gravel | SC | A-2-6(0) |

**Project No.** G10.1354.002 **Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

○ **Location:** Boring 6 **Depth:** 1-5 feet **Sample Number:** 5830-7

□ **Location:** Boring 6 **Depth:** 14 feet **Sample Number:** 5830-8

△ **Location:** Boring 6 **Depth:** 34 feet **Sample Number:** 5830-9

◇ **Location:** Boring 6 **Depth:** 49 feet **Sample Number:** 5830-10

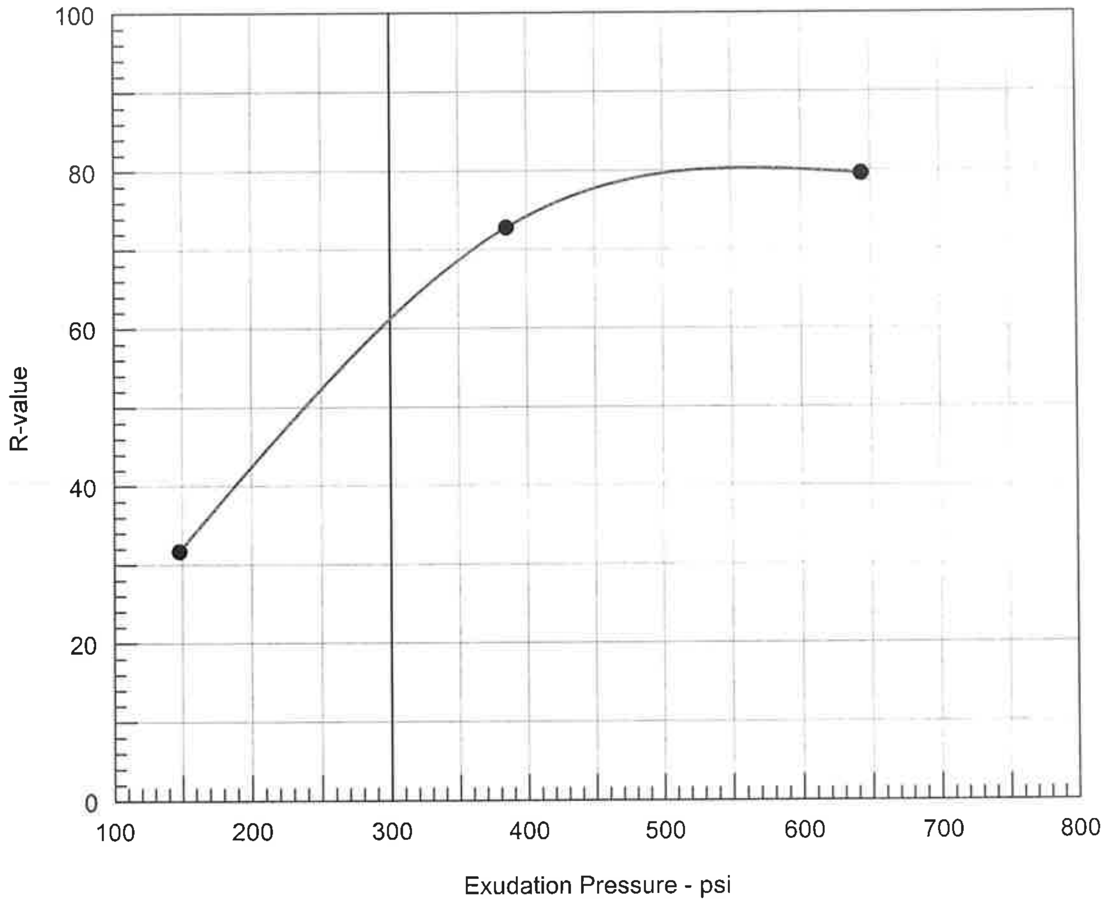
▽ **Location:** Boring 8 **Depth:** 4 feet **Sample Number:** 5971-3

**Remarks:**

# GEOCAL, INC.

**Figure** 10

# R-VALUE TEST REPORT




**Resistance R-Value and Expansion Pressure - AASHTO T 190**

| No. | Compact. Pressure psi | Density pcf | Moist. % | Expansion Pressure psf | Horizontal Press. psi @ 160 psi | Sample Height in. | Exud. Pressure psi | R Value | R Value Corr. |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 300 | 131.6 | 5.2 | 9 | 102 | 2.41 | 148 | 34 | 32 |
| 2 | 350 | 129.0 | 7.3 | 17 | 38 | 2.45 | 385 | 73 | 73 |
| 3 | 350 | 129.2 | 6.3 | 26 | 26 | 2.50 | 644 | 80 | 80 |
| | | | | | | | | | |

| Test Results | Material Description |
|---|---|
| **R-value at 300 psi exudation pressure =** 61 | poorly graded gravel with silty clay and sand, fill |

**Project No.:** G10.1354.002

**Project:** 6th Avenue over BNSF

**Location:** Boring 1

**Sample Number:** 5815-1 **Depth:** 1-10 feet

**Date:** 2/28/2012

R-VALUE TEST REPORT

**Geocal, Inc.**

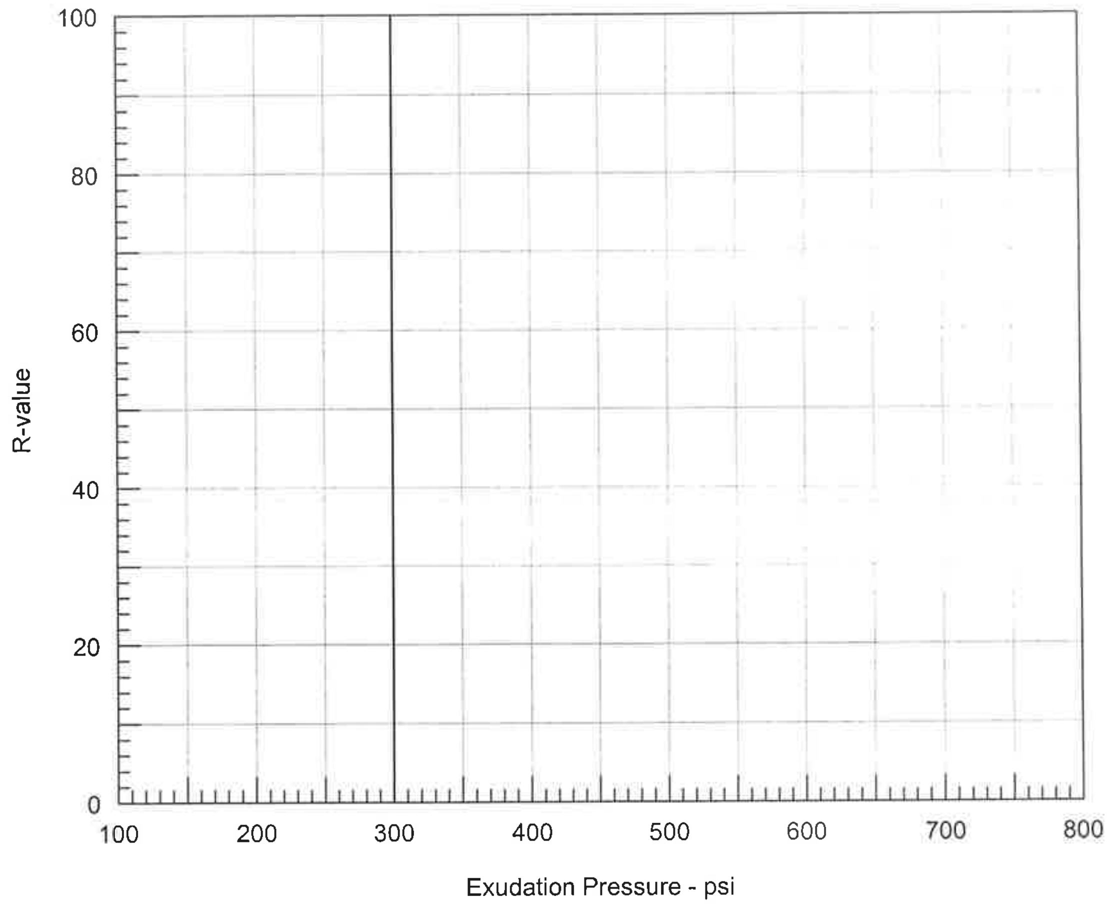
**Tested by:** H. Redzic

**Checked by:** G. Burgess, P.E.

**Remarks:**
Test performed in accordance with Colorado procedures CP-L 3101 & 3102

**Figure** 11

# R-VALUE TEST REPORT




**Resistance R-Value and Expansion Pressure - AASHTO T 190**

| No. | Compact. Pressure psi | Density pcf | Moist. % | Expansion Pressure psf | Horizontal Press. psi @ 160 psi | Sample Height in. | Exud. Pressure psi | R Value | R Value Corr. |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

| Test Results | Material Description |
|---|---|
| **R-value at 300 psi exudation pressure =** n/a | sandy lean clay, fill |

**Project No.:** G10.1354.002
**Project:** 6th Avenue over BNSF
**Location:** Boring 5
**Sample Number:** 5830-4 **Depth:** 1-5 feet
**Date:** 2/28/2012

**Tested by:** H. Redzic
**Checked by:** G. Burgess, P.E.

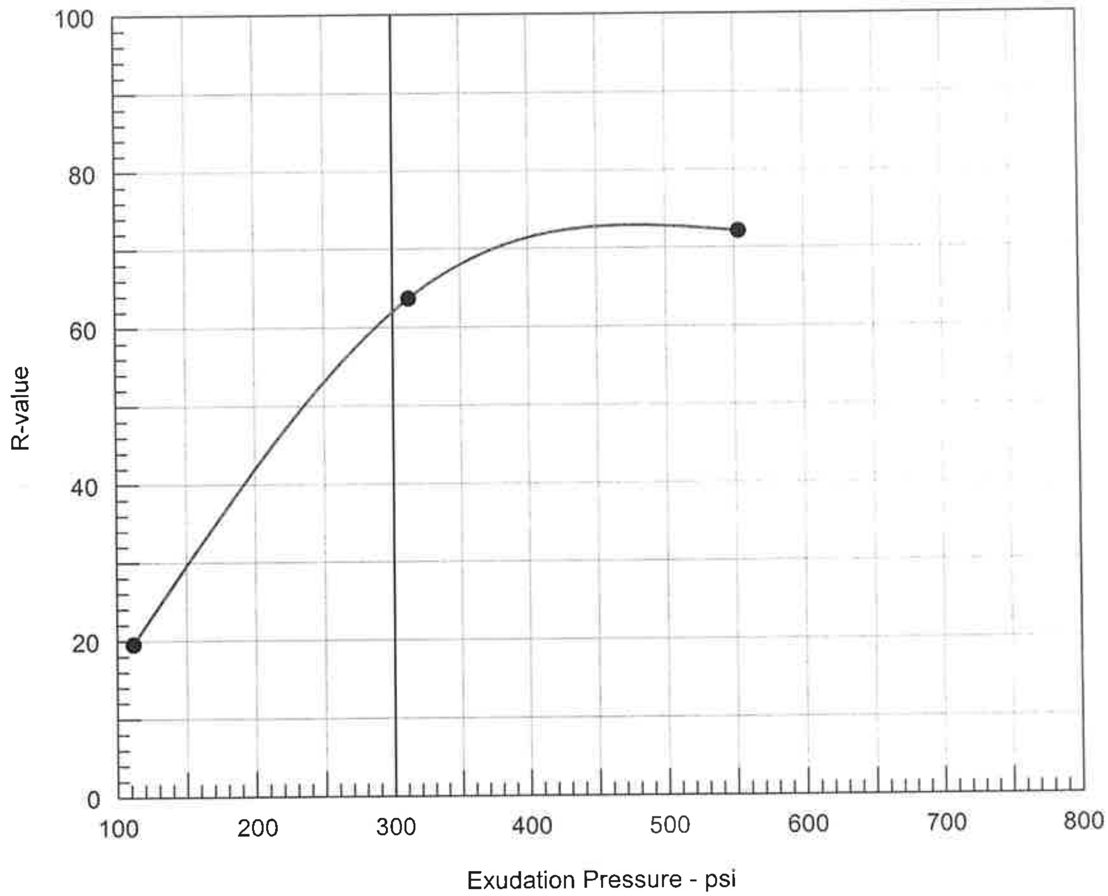
**Remarks:**
Sample extruded from under the mold during the exudation portion of test prior 800 psi.
R Value < 5.

R-VALUE TEST REPORT

**Geocal, Inc.**

**Figure** 12

# R-VALUE TEST REPORT




**Resistance R-Value and Expansion Pressure - AASHTO T 190**

| No. | Compact. Pressure psi | Density pcf | Moist. % | Expansion Pressure psf | Horizontal Press. psi @ 160 psi | Sample Height in. | Exud. Pressure psi | R Value | R Value Corr. |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 150 | 123.9 | 8.5 | 9 | 120 | 2.42 | 112 | 21 | 20 |
| 2 | 350 | 125.2 | 7.9 | 22 | 51 | 2.47 | 312 | 64 | 64 |
| 3 | 350 | 124.3 | 7.1 | 44 | 39 | 2.50 | 553 | 72 | 72 |
| | | | | | | | | | |

| Test Results | Material Description |
|---|---|
| **R-value at 300 psi exudation pressure =** 62 | clayey sand with gravel, fill |

**Project No.:** G10.1354.002

**Project:** 6th Avenue over BNSF

**Location:** Boring 6

**Sample Number:** 5830-7 **Depth:** 1-5 feet

**Date:** 2/28/2012

**Tested by:** H. Redzic

**Checked by:** G. Burgess, P.E.

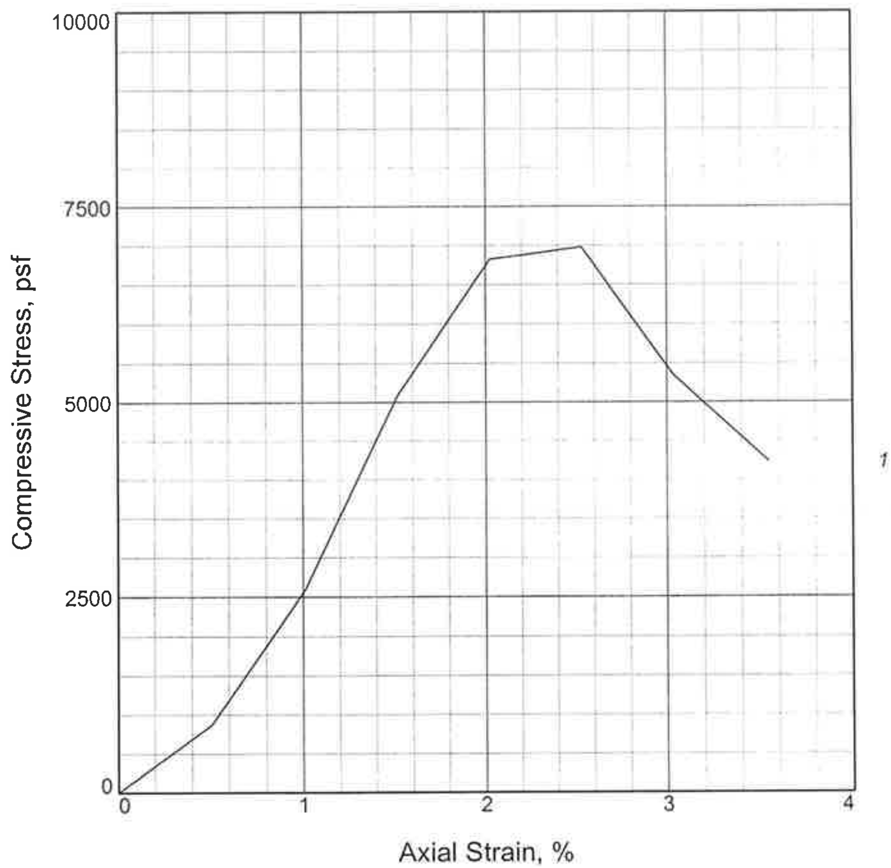
**Remarks:**
Test performed in accordance with colorado procedures CP-L 3101 & 3102.

R-VALUE TEST REPORT

**Geocal, Inc.**

**Figure** 13

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 6980 | | | |
| Undrained shear strength, psf | 3490 | | | |
| Failure strain, % | 2.5 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 16.2 | | | |
| Wet density, pcf | 134.0 | | | |
| Dry density, pcf | 115.3 | | | |
| Saturation, % | 98.6 | | | |
| Void ratio | 0.4346 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 3.95 | | | |
| Height/diameter ratio | 2.04 | | | |

**Description:** claystone bedrock

| LL = 42 | PL = 23 | PI = 19 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

**Project No.:** G10.1354.002

**Date Sampled:**

**Remarks:**

**Figure** 14

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

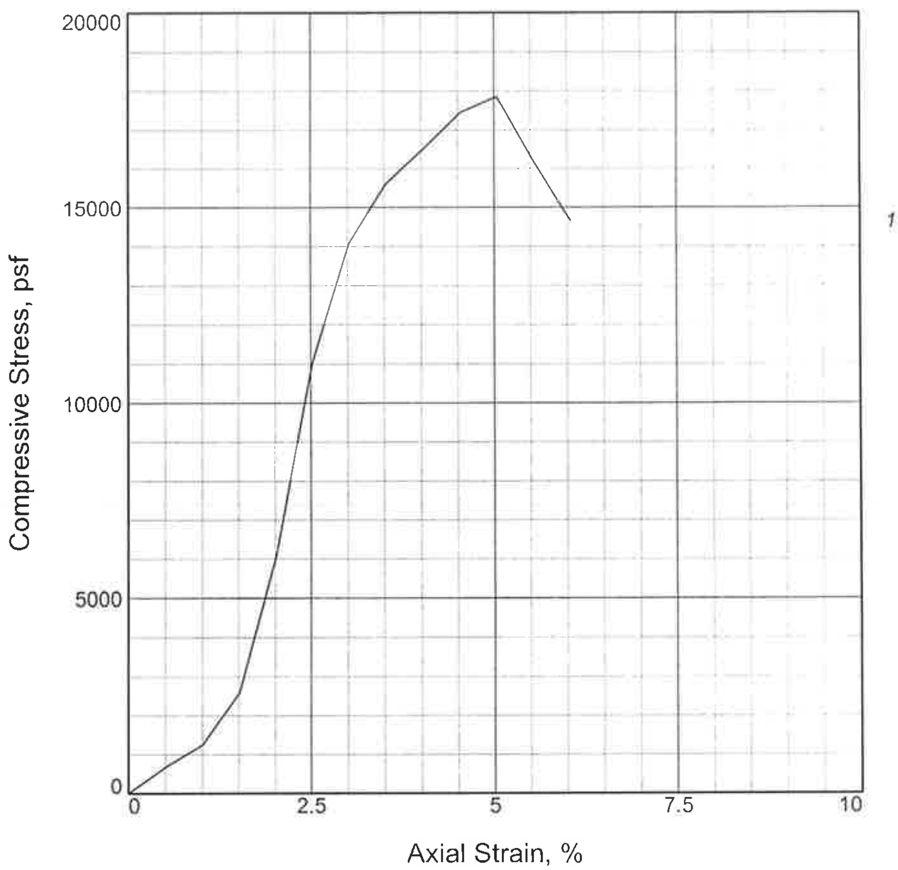
**Location:** Boring 1

**Sample Number:** 5815-5 **Depth:** 54 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 17857 | | | |
| Undrained shear strength, psf | 8928 | | | |
| Failure strain, % | 5.0 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 16.3 | | | |
| Wet density, pcf | 133.6 | | | |
| Dry density, pcf | 114.8 | | | |
| Saturation, % | 98.2 | | | |
| Void ratio | 0.4411 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 3.97 | | | |
| Height/diameter ratio | 2.05 | | | |

**Description:** claystone bedrock

| LL = 46 | PL = 24 | PI = 22 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

Project No.: G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 1

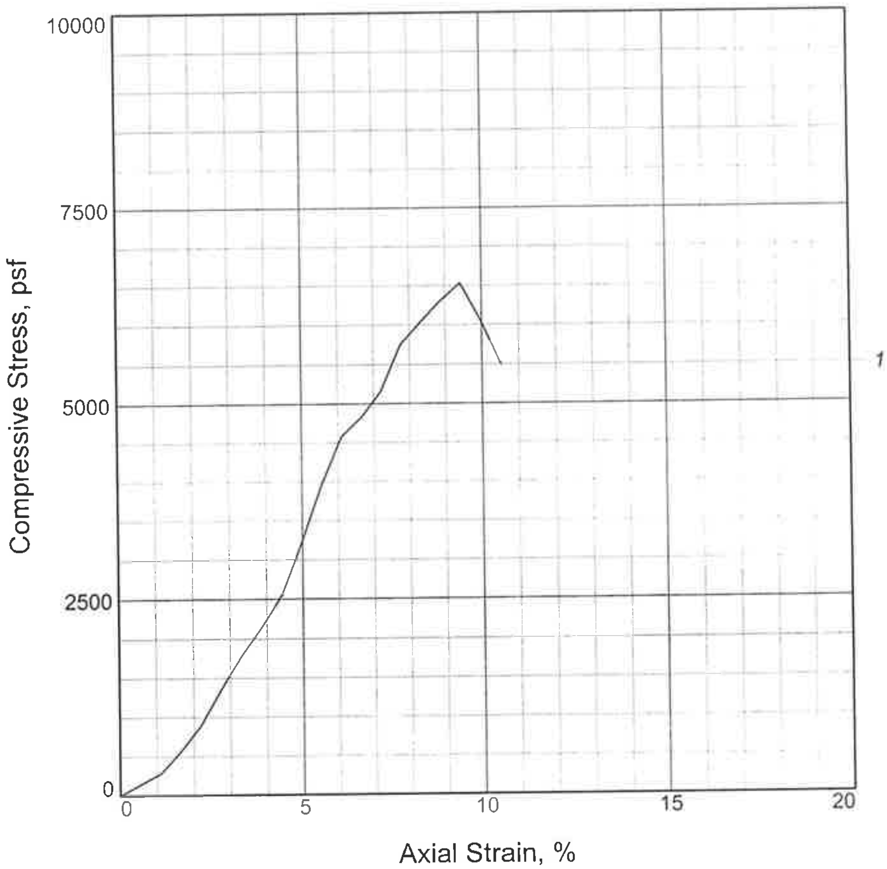
**Sample Number:** 5815-7 **Depth:** 64 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 15

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 6531 | | | |
| Undrained shear strength, psf | 3265 | | | |
| Failure strain, % | 9.4 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 15.9 | | | |
| Wet density, pcf | 121.1 | | | |
| Dry density, pcf | 104.5 | | | |
| Saturation, % | 72.2 | | | |
| Void ratio | 0.5828 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 3.61 | | | |
| Height/diameter ratio | 1.86 | | | |

**Description:** claystone bedrock

| LL = 43 | PL = 26 | PI = 17 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

**Project No.:** G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 2

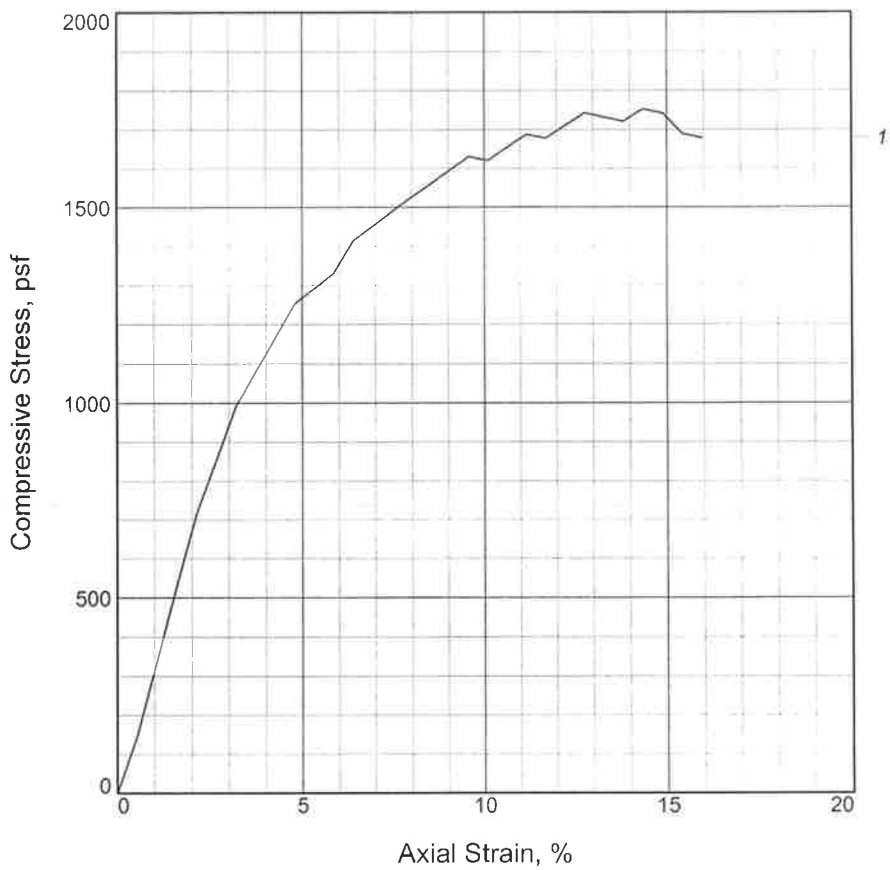
**Sample Number:** 5830-3 **Depth:** 54 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 16

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 1752 | | | |
| Undrained shear strength, psf | 876 | | | |
| Failure strain, % | 14.4 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 22.6 | | | |
| Wet density, pcf | 122.0 | | | |
| Dry density, pcf | 99.5 | | | |
| Saturation, % | 90.5 | | | |
| Void ratio | 0.6629 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 3.76 | | | |
| Height/diameter ratio | 1.94 | | | |

**Description:** sandy lean clay, fill

| LL = 44 | PL = 18 | PI = 26 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

Project No.: G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 3

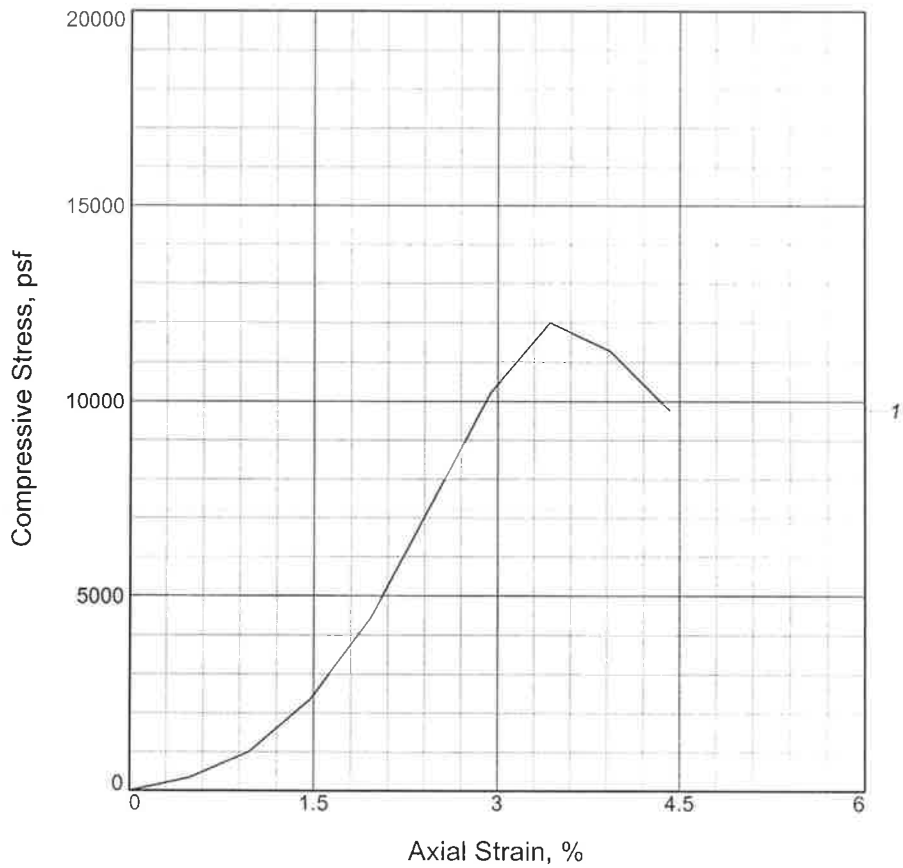
**Sample Number:** 5820-1 **Depth:** 4 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 17

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 11996 | | | |
| Undrained shear strength, psf | 5998 | | | |
| Failure strain, % | 3.4 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 17.3 | | | |
| Wet density, pcf | 129.8 | | | |
| Dry density, pcf | 110.7 | | | |
| Saturation, % | 92.6 | | | |
| Void ratio | 0.4947 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 4.08 | | | |
| Height/diameter ratio | 2.10 | | | |

**Description:** claystone bedrock

| | | | | |
|---|---|---|---|---|
| **LL =** 47 | **PL =** 26 | **PI =** 21 | **Assumed GS=** 2.65 | **Type:** |

**Project No.:** G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 3

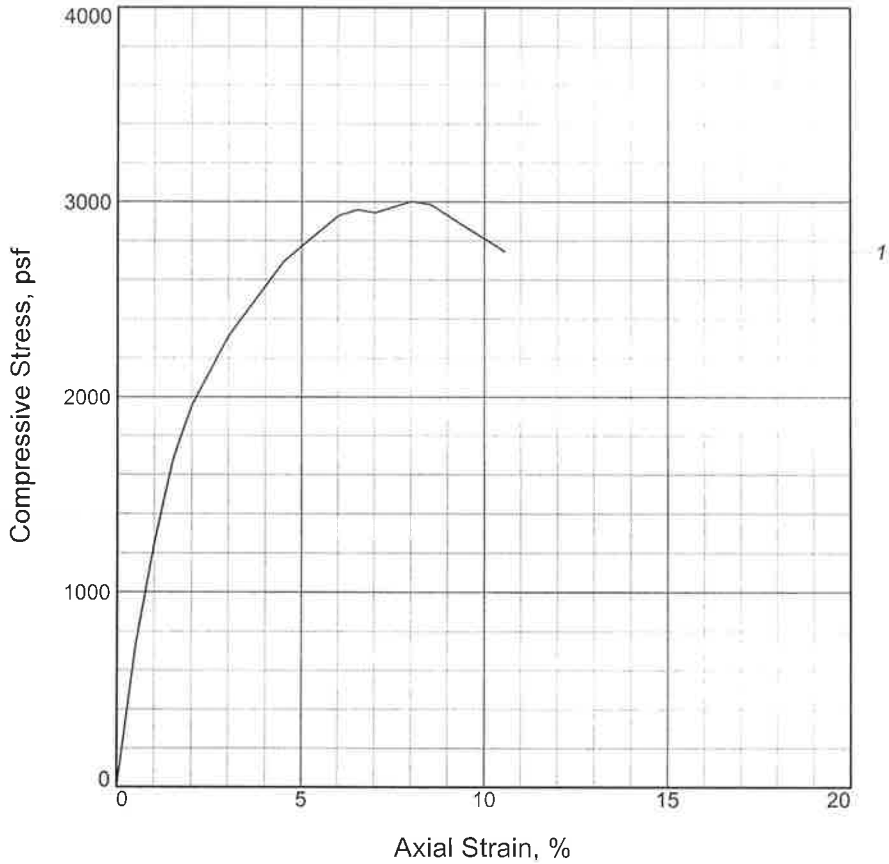
**Sample Number:** 5820-5 **Depth:** 59 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 18

# UNCONFINED COMPRESSION TEST




| | | | | |
|---|---|---|---|---|
| Sample No. | 1 | | | |
| Unconfined strength, psf | 3002 | | | |
| Undrained shear strength, psf | 1501 | | | |
| Failure strain, % | 8.0 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 21.0 | | | |
| Wet density, pcf | 124.2 | | | |
| Dry density, pcf | 102.7 | | | |
| Saturation, % | 90.8 | | | |
| Void ratio | 0.6113 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 3.98 | | | |
| Height/diameter ratio | 2.05 | | | |

**Description:** sandy lean clay, fill

| | | | | |
|---|---|---|---|---|
| **LL =** 47 | **PL =** 18 | **PI =** 29 | **Assumed GS=** 2.65 | **Type:** |

Project No.: G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 4

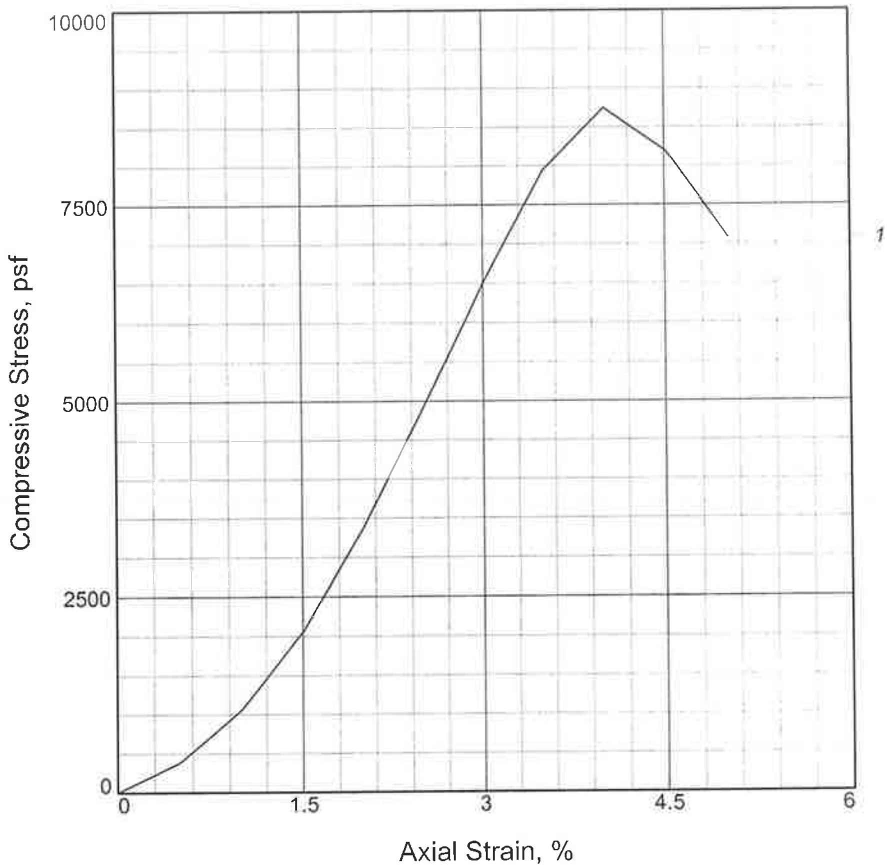
**Sample Number:** 5820-6 **Depth:** 9 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 19

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 8745 | | | |
| Undrained shear strength, psf | 4372 | | | |
| Failure strain, % | 4.0 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 18.5 | | | |
| Wet density, pcf | 129.5 | | | |
| Dry density, pcf | 109.3 | | | |
| Saturation, % | 95.5 | | | |
| Void ratio | 0.5132 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 3.99 | | | |
| Height/diameter ratio | 2.06 | | | |

**Description:** claystone bedrock

| LL = 40 | PL = 20 | PI = 20 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

Project No.: G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 4

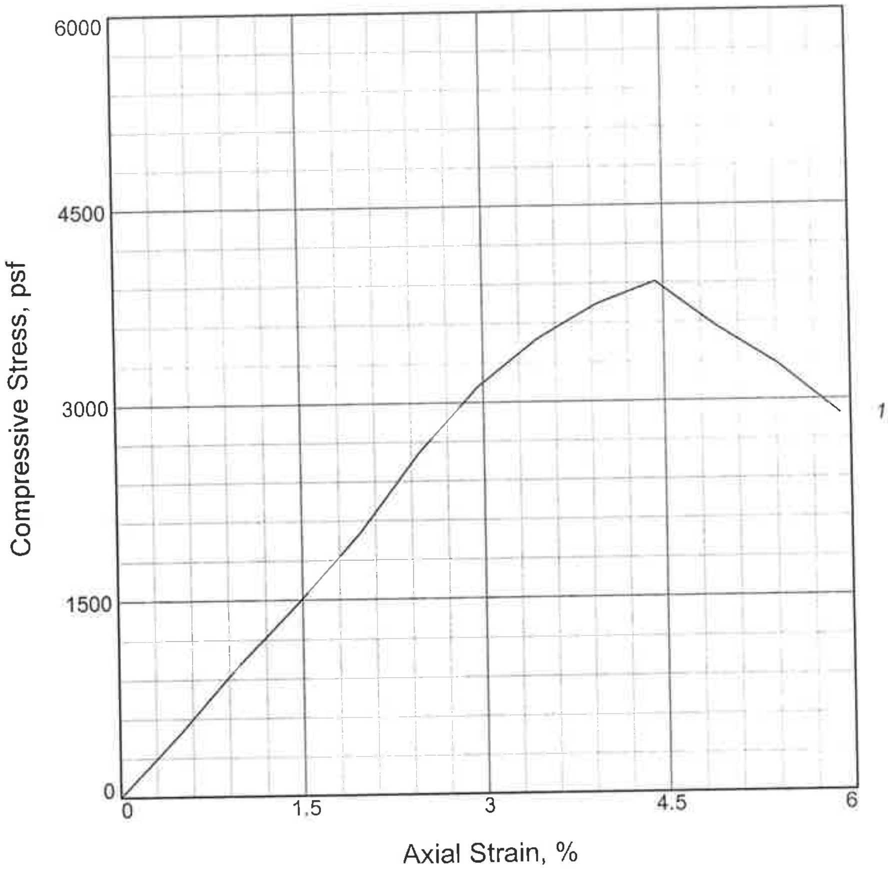
**Sample Number:** 5820-9 **Depth:** 49 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 20

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 3911 | | | |
| Undrained shear strength, psf | 1955 | | | |
| Failure strain, % | 4.4 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 17.5 | | | |
| Wet density, pcf | 128.7 | | | |
| Dry density, pcf | 109.5 | | | |
| Saturation, % | 90.8 | | | |
| Void ratio | 0.5110 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 4.06 | | | |
| Height/diameter ratio | 2.09 | | | |

**Description:** claystone bedrock

| LL = 48 | PL = 24 | PI = 24 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

**Project No.:** G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 6

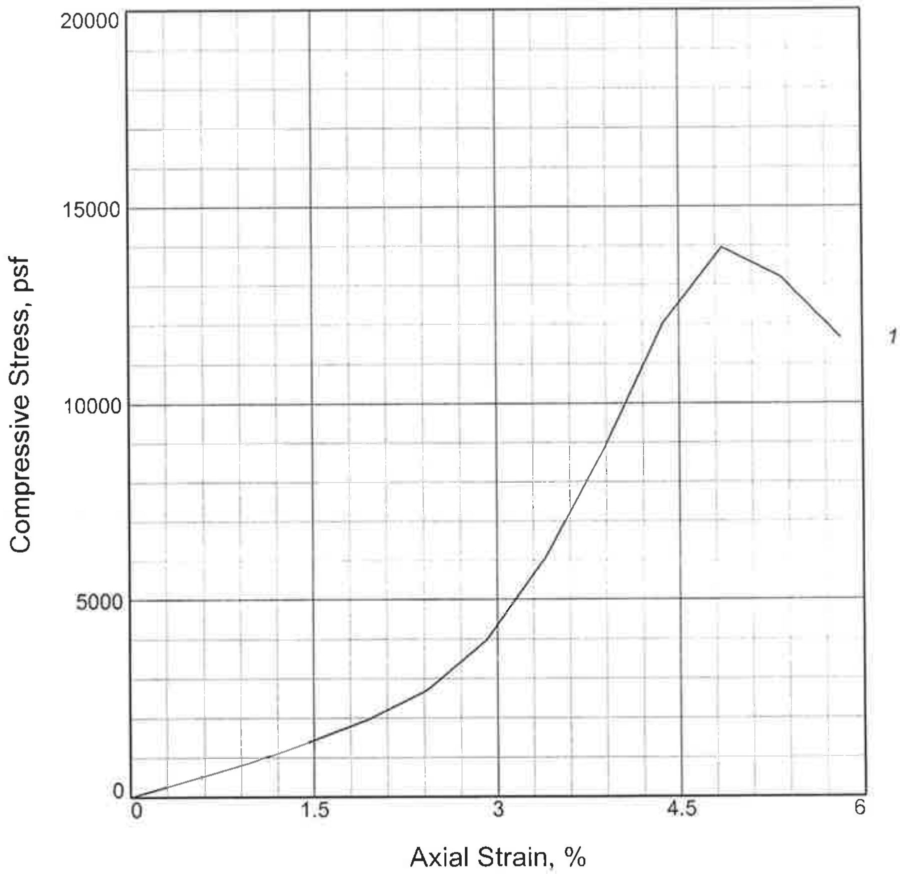
**Sample Number:** 5830-11 **Depth:** 54 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 21

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 13952 | | | |
| Undrained shear strength, psf | 6976 | | | |
| Failure strain, % | 4.9 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 18.2 | | | |
| Wet density, pcf | 126.9 | | | |
| Dry density, pcf | 107.3 | | | |
| Saturation, % | 89.3 | | | |
| Void ratio | 0.5413 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 4.12 | | | |
| Height/diameter ratio | 2.12 | | | |

**Description:** claystone bedrock

| **LL =** 45 | **PL =** 28 | **PI =** 17 | **Assumed GS=** 2.65 | **Type:** |
|---|---|---|---|---|

**Project No.:** G10.1354.002

**Date Sampled:**

**Remarks:**

**Figure** 22

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

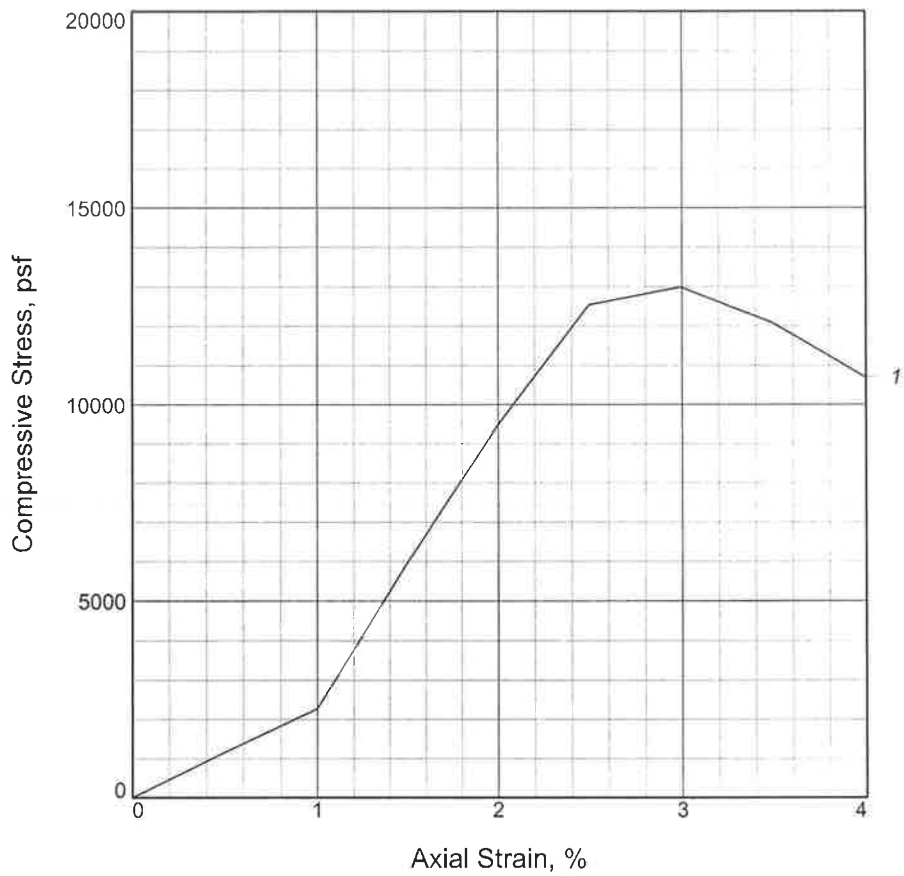
**Location:** Boring 7

**Sample Number:** 5971-2 **Depth:** 29

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 12996 | | | |
| Undrained shear strength, psf | 6498 | | | |
| Failure strain, % | 3.0 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 18.4 | | | |
| Wet density, pcf | 130.2 | | | |
| Dry density, pcf | 109.9 | | | |
| Saturation, % | 96.7 | | | |
| Void ratio | 0.5054 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 4.01 | | | |
| Height/diameter ratio | 2.07 | | | |

**Description:** claystone bedrock

| LL = 50 | PL = 25 | PI = 25 | Assumed GS= 2.65 | Type: |
|---|---|---|---|---|

Project No.: G10.1354.002

**Date Sampled:**

**Remarks:**

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 8

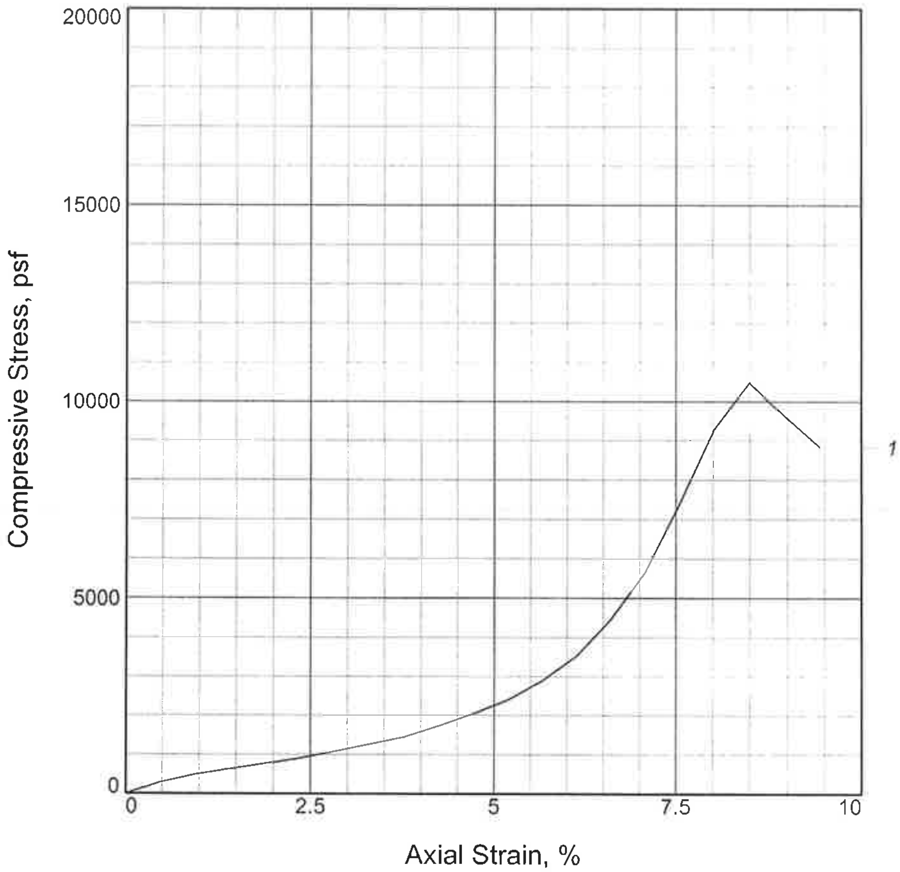
**Sample Number:** 5971-4 **Depth:** 24 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

**Figure** 23

# UNCONFINED COMPRESSION TEST




| Sample No. | 1 | | | |
|---|---|---|---|---|
| Unconfined strength, psf | 10476 | | | |
| Undrained shear strength, psf | 5238 | | | |
| Failure strain, % | 8.5 | | | |
| Strain rate, in./min. | 0.05 | | | |
| Water content, % | 16.2 | | | |
| Wet density, pcf | 125.6 | | | |
| Dry density, pcf | 108.1 | | | |
| Saturation, % | 81.0 | | | |
| Void ratio | 0.5303 | | | |
| Specimen diameter, in. | 1.94 | | | |
| Specimen height, in. | 4.24 | | | |
| Height/diameter ratio | 2.19 | | | |

**Description:** claystone bedrock

| **LL =** 41 | **PL =** 25 | **PI =** 16 | **Assumed GS=** 2.65 | **Type:** |
|---|---|---|---|---|

**Project No.:** G10.1354.002

**Date Sampled:**

**Remarks:**

**Figure** 24

**Client:** Wilson & Company

**Project:** 6th Avenue over BNSF

**Location:** Boring 8

**Sample Number:** 5971-5 **Depth:** 34 feet

UNCONFINED COMPRESSION TEST

GEOCAL, INC.

## TABLE 1
## SUMMARY OF LABORATORY TEST RESULTS

Project # **G10.1354.002**

Client: **Wilson& Company**
Project Name **6th Avenue over BNSF**

| Sample Location | | Natural Moisture Content (%) | Natural Dry Density (pcf) | Gradation | | Percent Passing No. 200 Sieve | Atterberg Limits | | Swell Pressure (psf) | Swell w/1or2 ksf Surcharge (%) | Unconfined Compressive Strength (psf) | R-Value at 300 psi Exudation Pressure | AASHTO Class (Group Index) | Soil or Bedrock Description |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Boring No. | Depth (feet) | | | Gravel (%) | Sand (%) | | Liquid Limit (%) | Plasticity Index (%) | | | | | | |
| 1 | 1-10 | | | 46 | 42 | 12 | 21 | 6 | | | | 60 | A-1-a | Poorly graded gravel with silty clay and sand, fill |
| 1 | 24 | 20.7 | 107 | | | | | | 0 | 0.1 | | | | Sandy lean clay, fill |
| 1 | 34 | 20.2 | 101 | | | | | | | | | | | Clayey sand with gravel, fill |
| 1 | 49 | | | 4 | 92 | 4 | NV | NP | | | | | A-1-b | Well-graded sand |
| 1 | 54 | 16.2 | 116 | | | | | | | | 6,980 | | | Claystone bedrock |
| 1 | 59 | 17.3 | 112 | | | | | | 1,030 | 0.8 | | | | Claystone bedrock |
| 1 | 64 | 16.3 | 115 | | | | | | | | 17,860 | | | Claystone bedrock |
| 2 | 4 | 20.4 | 106 | 1 | 49 | 50 | 37 | 21 | | | | | A-6(7) | Sandy lean clay, fill |
| 2 | 34 | 15.0 | 119 | 15 | 44 | 41 | 37 | 22 | | | | | A-6(4) | Clayey sand with gravel, fill |
| 2 | 54 | 15.9 | 105 | | | | | | | | 6,530 | | | Claystone bedrock |
| 3 | 4 | 22.6 | 100 | 2 | 35 | 63 | 44 | 26 | | | 1,750 | | A-7-6(14) | Sandy lean clay, fill |
| 3 | 9 | 15.9 | 109 | | | | | | | | | | | Silty, sandy clay with gravel, fill |
| 3 | 34 | 32.9 | 92 | 3 | 23 | 74 | 57 | 34 | 0 | 0.1 | | | A-7-6(25) | Fat clay with sand, fill |
| 3 | 44 | | | 24 | 62 | 14 | NV | NP | | | | | A-1-b | Silty sand with gravel |
| 3 | 59 | 17.3 | 111 | | | | | | | | 12,000 | | | Claystone bedrock |
| 4 | 9 | 21.0 | 103 | | | | | | | | 3,000 | | | Sandy lean clay, fill |
| 4 | 24 | 17.4 | 110 | | | | | | 0 | 0.0 | | | | Clayey sand with gravel, fill |
| 4 | 39 | | | 16 | 75 | 9 | NV | NP | | | | | A-1-b | Well-graded sand with silt and gravel |
| 4 | 49 | 18.5 | 110 | | | | | | | | 8,750 | | | Claystone bedrock |
| 4 | 54 | 16.3 | 112 | | | | | | 0 | 0.1 | | | | Sandstone bedrock |
| 5 | 1-5 | | | 4 | 44 | 52 | 40 | 24 | | | | <5 | A-6(9) | Sandy lean clay, fill |
| 5 | 44 | | | 17 | 78 | 5 | NV | NP | | | | | A-1-b | Well-graded sand with silt and gravel |
| 5 | 54 | 16.0 | 110 | | | | | | 0 | 0.0 | | | | Claystone bedrock |
| 6 | 1-5 | | | 26 | 53 | 21 | 26 | 10 | | | | 62 | A-2-4(0) | Clayey sand with gravel, fill |
| 6 | 14 | 22.2 | 104 | 0 | 39 | 61 | 41 | 25 | | | | | A-7-6(12) | Sandy lean clay, fill |
| 6 | 34 | 50.5 | 68 | 0 | 28 | 72 | 52 | 21 | | | | | A-7-5(16) | Elastic silt with sand |
| 6 | 49 | 15.6 | 110 | 0 | 9 | 91 | 43 | 20 | 0 | 0.5 | | | | Claystone bedrock |
| 6 | 54 | 17.5 | 110 | | | 91 | 48 | 24 | | | 3,910 | | | Claystone bedrock |
| 7 | 2 | 19.5 | 94 | | | 46 | 43 | 23 | | | | | A-7-6(6) | Clayey sand with gravel |
| 7 | 29 | 18.2 | 107 | | | 80 | 45 | 17 | | | 13,950 | | | Claystone bedrock |
| 8 | 4 | | | 23 | 55 | 22 | 23 | 11 | | | | | A-2-6-(0) | Clayey sand with gravel |
| 8 | 24 | 18.4 | 110 | | | 95 | 50 | 25 | | | 13,000 | | | Claystone bedrock |
| 8 | 34 | 16.2 | 108 | | | 81 | 41 | 16 | | | 10,480 | | | Claystone bedrock |

Project #: G10.1354.002

## TABLE 2
## SUMMARY OF LABORATORY CHEMICAL TEST RESULTS

Client: **Wilson & Company**
Project Name: **6th Avenue over BNSF**

| Sample Location | | Natural Moisture Content (%) | Natural Dry Density (pcf) | Water Soluble Sulfates (%) | Laboratory Resistivity (ohm-cm) | pH | Chloride Water Soluble (%) | Sulfide | AASHTO Class. (Group Index) | Soil or Bedrock Description |
|---|---|---|---|---|---|---|---|---|---|---|
| Boring No. | Depth (feet) | | | | | | | | | |
| 1 | 1-10 | | | 0.07 | 1,300 | 6.9 | 0.0256 | | A-1-a | Poorly graded gravel with silty clay and sand, fill |
| 1 | 34 | 20.2 | 101 | 0.06 | 1,200 | 6.8 | 0.0180 | | | Clayey sand with gravel, fill |
| 1 | 49 | | | 0.07 | 4,500 | 7.0 | 0.0019 | | A-1-b | Well-graded sand |
| 2 | 4 | 20.4 | 106 | 0.07 | 420 | 6.3 | 0.0696 | Positive | A-6(7) | Sandy lean clay, fill |
| 2 | 34 | 15.0 | 119 | 0.07 | 1,600 | 6.8 | 0.0021 | Positive | A-6(4) | Clayey sand with gravel, fill |
| 3 | 9 | 15.9 | 109 | 0.07 | 790 | 7.6 | 0.0196 | Trace | | Silty, sandy clay with gravel, fill |
| 3 | 44 | | | 0.05 | 3,700 | 7.4 | 0.0030 | Negative | A-1-b | Silty sand with gravel |
| 4 | 9 | 21.0 | 103 | 0.18 | 490 | 6.9 | 0.0139 | Positive | | Sandy lean clay |
| 5 | 54 | 16.0 | 110 | 0.06 | 680 | 6.7 | 0.0015 | Positive | | Claystone bedrock |
| 6 | 14 | 22.2 | 104 | 0.13 | 460 | 6.7 | 0.0349 | Trace | A-7-6(12) | Sandy lean clay |
| 6 | 34 | 50.5 | 68 | 0.08 | 1,400 | 6.8 | 0.0079 | Positive | A-7-5(16) | Elastic silt with sand |
| 7 | 2 | 19.5 | 94 | 0.02 | 170 | 5.7 | 0.2181 | Negative | A-7-6(6) | Clayey sand with gravel |
| 8 | 4 | | | 0.03 | 5,000 | 5.5 | 0.0022 | Negative | A-2-6(0) | Clayey sand with gravel |

# Appendix A

## Laboratory Test Results

## Colorado Analytical Laboratories, Inc.

Colorado Analytical Laboratories, Inc.

# Analytical Results

***TASK NO: 111102009***

**Report To:** Husein Redzic
**Company:** Geocal
7290 S. Fraser St
Centennial CO 80112

**Bill To:** Husein Redzic
**Company:** Geocal
7290 S. Fraser St
Centennial CO 80112

**Task No.:** 111102009
**Client PO:** 2976
**Client Project:** 6th Avenue over BNSF G10.1354.002

**Date Received:** 11/2/11
**Date Reported:** 11/9/11
**Matrix:** Soil - Geotech

**Customer Sample ID** B-1 @ 49 5815-4
**Sample Date/Time:**
**Lab Number:** 111102009-01

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0019 %** | AASHTO T291-91/ ASTM D4327 |

**Customer Sample ID** B-1 @ 34 5815-3
**Sample Date/Time:**
**Lab Number:** 111102009-02

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0180 %** | AASHTO T291-91/ ASTM D4327 |

**Customer Sample ID** B-1 @ 1-10 5815-1
**Sample Date/Time:**
**Lab Number:** 111102009-03

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0256 %** | AASHTO T291-91/ ASTM D4327 |

***Abbreviations/ References:***

*AASHTO - American Association of State Highway and Transportation Officials.*
*ASTM - American Society for Testing and Materials.*
*ASA - American Society of Agronomy.*
*DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.*

Steve Nielson

**DATA APPROVED FOR RELEASE BY**



**240 South Main Street / Brighton, CO 80601-0507 / 303-659-2313**
**Mailing Address: P.O. Box 507 / Brighton, CO 80601-0507 / Fax: 303-659-2315**

Colorado Analytical Laboratories, Inc.

# Analytical Results

***TASK NO: 111129004***

**Report To:** Husein Redzic
**Company:** Geocal
7290 S. Fraser St
Centennial CO 80112

**Bill To:** Husein Redzic
**Company:** Geocal
7290 S. Fraser St
Centennial CO 80112

**Task No.:** 111129004
**Client PO:** 2993-1354
**Client Project:** 6th Ave. over BNSF G10.1354.002

**Date Received:** 11/29/11
**Date Reported:** 12/7/11
**Matrix:** Soil - Geotech

**Customer Sample ID** 5820-4 B-3 @ 44
**Sample Date/Time:**
**Lab Number:** 111129004-01

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0030 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Negative** | AWWA C105 |

**Customer Sample ID** 5830-2 B-2 @ 34
**Sample Date/Time:**
**Lab Number:** 111129004-02

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0021 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Positive** | AWWA C105 |

**Customer Sample ID** 5830-1 B-2 @ 4
**Sample Date/Time:**
**Lab Number:** 111129004-03

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0696 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Positive** | AWWA C105 |

**Customer Sample ID** 5830-8 B-6 @ 14
**Sample Date/Time:**
**Lab Number:** 111129004-04

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0349 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Trace** | AWWA C105 |

***Abbreviations/ References:***

*AASHTO - American Association of State Highway and Transportation Officials.*
*ASTM - American Society for Testing and Materials.*
*ASA - American Society of Agronomy.*
*DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.*

Shane Nielson

**DATA APPROVED FOR RELEASE BY**



240 South Main Street / Brighton, CO 80601-0507 / 303-659-2313
Mailing Address: P.O. Box 507 / Brighton, CO 80601-0507 / Fax: 303-659-2315

Colorado Analytical Laboratories, Inc.

# Analytical Results

***TASK NO: 111129004***

**Report To:** Husein Redzic
**Company:** Geocal

**Bill To:** Husein Redzic
**Company:** Geocal

**Task No.:** 111129004
**Client PO:** 2993-1354
**Client Project:** 6th Ave. over BNSF G10.1354.002

**Date Received:** 11/29/11
**Date Reported:** 12/7/11
**Matrix:** Soil - Geotech

**Customer Sample ID** 5830-9 B-6 @ 34
**Sample Date/Time:**
**Lab Number:** 111129004-05

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0079 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Positive** | AWWA C105 |

**Customer Sample ID** 5830-6 B-5 @ 54
**Sample Date/Time:**
**Lab Number:** 111129004-06

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0015 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Positive** | AWWA C105 |

**Customer Sample ID** 5820-2 B-3 @ 9
**Sample Date/Time:**
**Lab Number:** 111129004-07

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0196 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Trace** | AWWA C105 |

**Customer Sample ID** 5820-6 B-4 @ 9
**Sample Date/Time:**
**Lab Number:** 111129004-08

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0139 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Positive** | AWWA C105 |

***Abbreviations/ References:***

*AASHTO - American Association of State Highway and Transportation Officials.*
*ASTM - American Society for Testing and Materials.*
*ASA - American Society of Agronomy.*
*DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.*

Shane Nielson

**DATA APPROVED FOR RELEASE BY**



240 South Main Street / Brighton, CO 80601-0507 / 303-659-2313
Mailing Address: P.O. Box 507 / Brighton, CO 80601-0507 / Fax: 303-659-2315

111129004

Colorado Analytical Laboratories, Inc.

# Analytical Results

***TASK NO: 120229001***

**Report To:** Husein Redzic
**Company:** Geocal
7290 S. Fraser St
Centennial CO 80112

**Bill To:** Husein Redzic
**Company:** Geocal
7290 S. Fraser St
Centennial CO 80112

**Task No.:** 120229001
**Client PO:** 3062-1354
**Client Project:** 6th Ave. over BNSF G10.1354.000

**Date Received:** 2/29/12
**Date Reported:** 3/2/12
**Matrix:** Soil - Geotech

**Customer Sample ID** Geo 8 @ 4 Ft.
**Sample Date/Time:**
**Lab Number:** 120229001-01

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.0022 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Negative** | AWWA C105 |

**Customer Sample ID** Geo 7 @ 2 Ft.
**Sample Date/Time:**
**Lab Number:** 120229001-02

| Test | Result | Method |
|---|---|---|
| Chloride - Water Soluble | **0.2181 %** | AASHTO T291-91/ ASTM D4327 |
| Sulfide | **Negative** | AWWA C105 |

***Abbreviations/ References:***
*AASHTO - American Association of State Highway and Transportation Officials.*
*ASTM - American Society for Testing and Materials.*
*ASA - American Society of Agronomy.*
*DIPRA - Ductile Iron Pipe Research Association Handbook of Ductile Iron Pipe.*

Shane Nielson

**DATA APPROVED FOR RELEASE BY**



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# Appendix B

## Preliminary Pavement Design Data and Analysis

# Annual Average Daily Traffic (AADT) Volumes for Highway 006G

| Route | Ref Pt | End Ref Pt | Length (Miles) | Annual Average Daily Traffic | AADT Year | AADT Single Trucks | AADT Comb Trucks | Percent Trucks | Design Hour Volume (% of AADT) | Daily Vehicle Miles Traveled | Segment Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 006G | 282.333 | 283.469 | 1.106 | 115,000 | 2010 | 2750 | 1250 | 3.50 | 10 | 127,190 | SHERIDAN BLVD INTERCHANGE STR (F-16-FL) - JCT SH 095A N AND S - RD N AND S (SHERIDAN BLVD) - OVERPASS SEPARATION - LEAVE JEFFERSON COUNTY - LEAVE LAKEWOOD CITY LIMITS |
| 006G | 284.187 | 284.748 | 0.560 | 141,000 | 2010 | 2400 | 1700 | 2.90 | 8 | 78,960 | MAJOR STR (F-16-EN) - RD N AND S (BRYANT ST) OVERPASS SEPARATION |

# Design Lane ESAL Calculations

| 6th Ave. Over BNSF | | | Vehicle Type/Classification (%) | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Passenger Vehicles | Single Unit | Combination Unit | | | |
| **Vehicle Type Load Factor (Flexible)** | | | 0.003 | 0.249 | 1.087 | | | |
| **Vehicle Type Load Factor (Rigid)** | | | 0.003 | 0.285 | 1.692 | | | |
| | | | Number of Lanes = 6 | | | % in Design Lane 30% | | |
| *Percent of types* | *Year* | *100.0%* | *96.5%* | *2.4%* | *1.1%* | | | |
| *"Current" ADT (CDOT provided)* | *2010* | *115,000* | *110,975* | *2,760* | *1,265* | | | |
| *"Future" ADT (Projected)* | *2035* | *240,784* | Calculated Average Annual Increase | | | 3.00% | 25 | Years between 'Current' & 'Future' ADT |
| Construction Year ADT | 2014 | 129,433 | 124,903 | 3,106 | 1,424 | | | |
| End Year ADT (Flexible) | 2034 | 233,771 | 225,589 | 5,611 | 2,571 | | | |
| End Year ADT (Rigid) | 2044 | 314,169 | 303,173 | 7,540 | 3,456 | | | |
| **20-Yr Design ADT** | **2024** | **181,603** | **175,246** | **4,359** | **1,998** | | | |
| **Roadway ESAL (Flexible)** | **2029** | **27,615,571** | **3,837,887** | **7,923,354** | **15,854,330** | | | |
| **30-Yr Design ADT** | | **221,801** | **214,038** | **5,323** | **2,440** | | | |
| **Roadway ESAL (Rigid)** | | **45,899,838** | **4,687,432** | **11,074,502** | **30,137,904** | | | |
| **Design Lane ESAL (Flex)** | | ***8,284,671*** | | | | | | |
| **Design Lane ESAL (Rigid)** | | ***13,769,951*** | | | | | | |

1993 AASHTO Pavement Design

# DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Walter Zitz

## Flexible Structural Design Module

6th Avenue over BNSF Bridge Replacement
Approach Pavements
Design Life: 20 Years (Asphalt)
Assume R-50 Subgrade

## Flexible Structural Design

| | |
|---|---|
| 18-kip ESALs Over Initial Performance Period | 8,284,671 |
| Initial Serviceability | 4.5 |
| Terminal Serviceability | 2.5 |
| Reliability Level | 95 % |
| Overall Standard Deviation | 0.44 |
| Roadbed Soil Resilient Modulus | 13,168 psi |
| Stage Construction | 1 |
| Calculated Design Structural Number | 4.02 in |

## Specified Layer Design

| Layer | Material Description | Struct Coef. (Ai) | Drain Coef. (Mi) | Thickness (Di)(in) | Width (ft) | Calculated SN (in) |
|---|---|---|---|---|---|---|
| 1 | HMA | 0.44 | 1 | 7.5 | - | 3.30 |
| 2 | Class 6 ABC | 0.12 | 1 | 6 | - | 0.72 |
| Total | - | - | - | 13.50 | - | 4.02 |

## Layered Thickness Design

Thickness precision Actual

| Layer | Material Description | Struct Coef. (Ai) | Drain Coef. (Mi) | Spec Thickness (Di)(in) | Min Thickness (Di)(in) | Elastic Modulus (psi) | Width (ft) | Calculated Thickness (in) | Calculated SN (in) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | HMA | 0.44 | 1 | - | - | - | - | 9.14 | 4.02 |
| Total | - | - | - | - | - | - | - | 9.14 | 4.02 |

# Rigid Pavement Design - Based on AASHTO Supplemental Guide

Reference: *LTPP DATA ANALYSIS - Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*

## I. General

Agency: CDOT
Street Address: 6th Avenue over BNSF
City: Denver
State: Colorado

Project Number:

ID: 6th Avenue over BNSF

Description: Design of approach pavements to Bridge Structure

Location: Region 6

## II. Design

**Serviceability**

Initial Serviceability, P1: 4.5
Terminal Serviceability, P2: 2.5

**PCC Properties**

28-day Mean Modulus of Rupture, $(S'_c)'$: 650 psi
Elastic Modulus of Slab, $E_c$: 3,400,000 psi
Poisson's Ratio for Concrete, m: 0.15

**Base Properties**

Elastic Modulus of Base, $E_b$: 15,000 psi
Design Thickness of Base, $H_b$: 6.0 in
Slab-Base Friction Factor, f: 1.4

**Reliability and Standard Deviation**

Reliability Level (R): 95.0 %
Overall Standard Deviation, $S_0$: 0.34

**Climatic Properties**

Mean Annual Wind Speed, WIND: 8.8 mph
Mean Annual Air Temperature, TEMP: 50.3 °F
Mean Annual Precipitation, PRECIP: 15.3 in

**Subgrade k-Value**

175 psi/in

**Design ESALs**

13.8 million

Pavement Type, Joint Spacing (L)

- (●) JPCP
- ( ) JRCP
- ( ) CRCP

Joint Spacing: 15.0 ft

JPCP

Effective Joint Spacing: 180 in

Edge Support

- (●) Conventional 12-ft wide traffic lane
- ( ) Conventional 12-ft wide traffic lane + tied PCC
- ( ) 2-ft widened slab w/conventional 12-ft traffic lane

Edge Support Factor: 1.00

Sensitivity Analysis

Slab Thickness used for Sensitivity Analysis: 10.84 in

- ( ) Modulus of Rupture
- ( ) Elastic Modulus (Slab)
- ( ) Elastic Modulus (Base)
- (●) Base Thickness
- ( ) k-Value
- ( ) Joint Spacing
- ( ) Reliability
- ( ) Standard Deviation

**Calculated Slab Thickness for Above Inputs: 10.84 in**

## Faulting

### DOWELED PAVEMENT

| | | |
|---|---|---|
| Dowel Diameter: | 1.50 | in |
| $K_d$: | 1,500,000 | psi/in |
| $E_s$: | 29,000,000 | psi |

Base/Slab Frictional Restraint

- ○ Stabilized Base
- ◉ Aggregate Base or LCB w/ bond breaker

| | | |
|---|---|---|
| ALPHA: | 0.000006 | /°F |
| TRANGE: | 120.0 | °F |
| e: | 0.00015 | strain |
| D: | 10.43 | in |
| P: | 9,000 | lbf |
| T: | 0.45 | |

Base Type

- ○ Stabilized Base
- ◉ Unstabilized Base

| | | |
|---|---|---|
| FI: | 660 | °F-days |
| CESAL: | 13.77 | million |
| Age: | 22.0 | years |
| $C_d$: | 1.00 | |

**Faulting (doweled)**

**0.06** **in**

Faulting Check - **PASS**

### NONDOWELED PAVEMENT

| | | |
|---|---|---|
| Days90: | 56 | days |
| D: | 10.84 | in |

Base Type

- ○ Stabilized Base
- ◉ Unstabilized Base

| | | |
|---|---|---|
| FI: | 477 | °F-days |
| CESAL: | 13.77 | million |
| Age: | 20.0 | years |
| $C_d$: | 1.00 | |

**Faulting (nondoweled)**

**in**

Faulting Check -

Recommended critical mean joint faulting levels for design (Table 28)

| Joint Spacing | Critical Mean Joint Faulting |
|---|---|
| < 25 ft | 0.06 in |
| > 25 ft | 0.13 in |

**Note: Joint load position stress checks need to be performed only for nondoweled pavements**

Only two numbers need to be entered in this sheet:

**Temperature gradient**

**Tensile stress at top of slab**

**Step 1:**

Total Negative Temperature Differential

Slab Thickness: 10.84 in

Total Negative Temperature Differential: -6.3 °F

Construction Curling and Moisture Gradient Temperature Differential

**Enter temperature gradient:** 2.0 °F/in (enter positive value from below)

For temperature gradient use:

| | | |
|---|---|---|
| **Wet Climate:** | 0 to 2 °F/in | (Annual Precipitation >= 30 in or Thornthwaite Moisture Index > 0) |
| **Dry Climate:** | 1 to 3 °F/in | (Annual Precipitation < 30 in or Thornthwaite Moisture Index < 0) |

Total Effective Negative Temp. Differential: -28.0 °F

**Step 2:**

Use one or more of the following charts to estimate the tensile stress at top of slab. Note that the charts show the variation of tensile stress with negative temperature differential for slab thicknesses ranging from 7 to 13 in. These are plotted for a base course thickness of 6 in. The six charts represent three k-values (100, 250 and 500 psi/in) and two values for the elastic modulus of the base (25,000 psi and 1,000,000 psi). Use judgment to extrapolate the value of the tensile stress at the top of the slab from these charts.

**Enter Tensile Stress at Top of Slab:** 175 psi (use charts below)

**Step 3:**

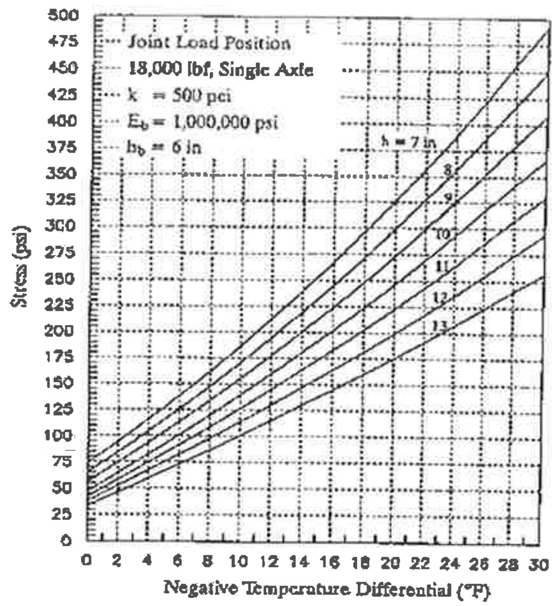
Compare the above tensile stress with the maximum tensile stress at the bottom of the slab for which the slab is designed. For the given inputs and the above thickness, this value is

197 psi

The slab is designed for a tensile stress of 197 psi.

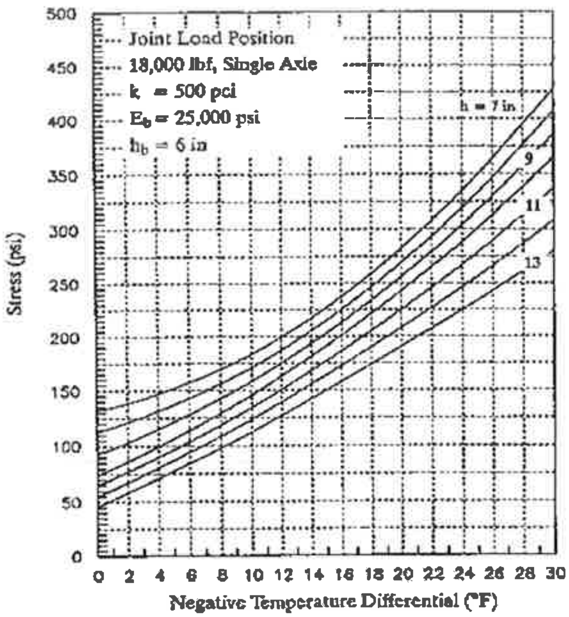
If the tensile stress at the top of the slab (obtained from the charts below and entered above) is less than the design stress, the design is acceptable. If the check fails, new inputs have to be provided.

| | |
|---|---|
| **Corner Break Check:** | **PASS** |



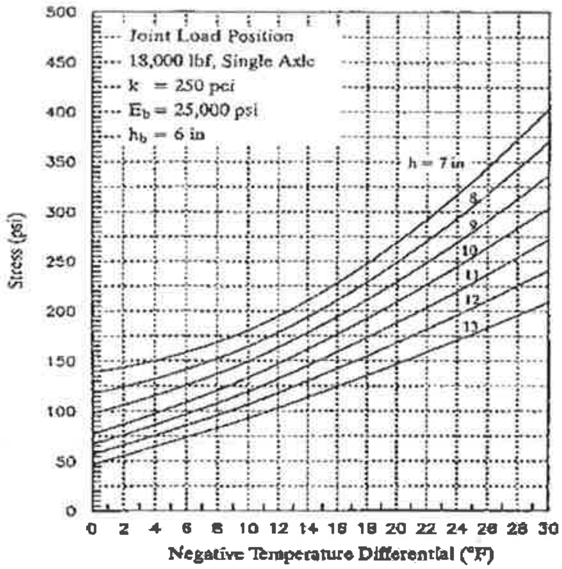

1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 59. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for high-strength base and stiff subgrade.



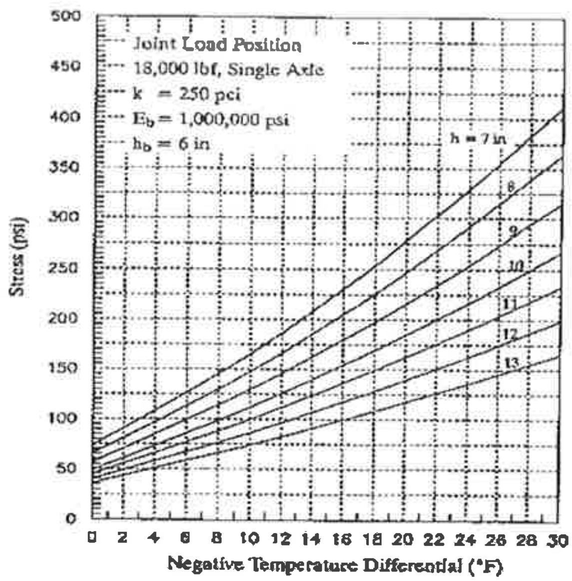

1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 58. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for aggregate base and stiff subgrade.



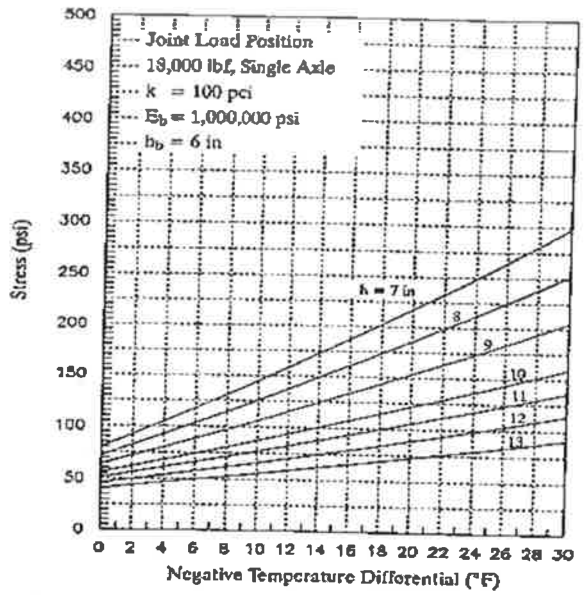

1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 56. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for aggregate base and medium subgrade.



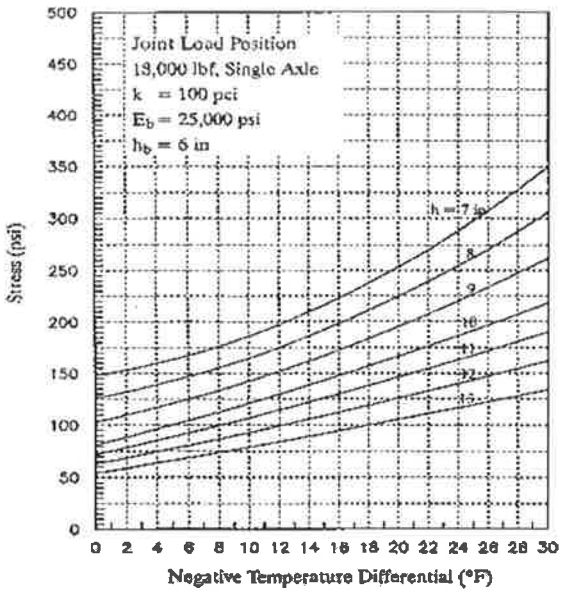

1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

**Figure 57. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for high-strength base and medium subgrade.**




1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 55. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for high-strength base and soft subgrade.




1 lbf = 4.45 N, 1 pci = 0.271 kPa/mm, 1 psi = 6.89 kPa, 1 in = 25.4 mm, °C = (°F - 32)/1.8

Figure 54. Tensile stress at top of slab for joint loading position, negative temperature differential, and full friction, for aggregate base and soft subgrade.

# Rigid Pavement Design - Based on AASHTO Supplemental Guide

Reference: *LTPP DATA ANALYSIS - Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*

## Results

Project #
Description: Design of approach pavements to Bridge Structure

Location: Region 6

### Slab Thickness Design

| | | |
|---|---|---|
| Pavement Type | JPCP | |
| 18-kip ESALs Over Initial Performance Period (million) | 13.77 | million |
| Initial Serviceability | 4.5 | |
| Terminal Serviceability | 2.5 | |
| 28-day Mean PCC Modulus of Rupture | 650 | psi |
| Elastic Modulus of Slab | 3,400,000 | psi |
| Elastic Modulus of Base | 15,000 | psi |
| Base Thickness | 6.0 | in. |
| Mean Effective k-Value | 175 | psi/in |
| Reliability Level | 95 | % |
| Overall Standard Deviation | 0.34 | |
| Calculated Design Thickness | **10.84** | in |

### Temperature Differential

| | | |
|---|---|---|
| Mean Annual Wind Speed | 8.8 | mph |
| Mean Annual Air Temperature | 50.3 | $^oF$ |
| Mean Annual Precipitation | 15.3 | in |
| Maximum Positive Temperature Differential | **8.28** | $^oF$ |

### Modulus of Subgrade Reaction

| Period | Description | Subgrade k-Value, psi |
|---|---|---|

| | | |
|---|---|---|
| Seasonally Adjusted Modulus of Subgrade Reaction | | psi/in |
| Modulus of Subgrade Reaction Adjusted for Rigid Layer and Fill Section | **190** | psi/in |

## Traffic

| | | |
|---|---|---|
| Performance Period | | years |
| Two-Way ADT | | |
| Number of Lanes in Design Direction | | |
| Percent of All Trucks in Design Lane | | |
| Percent Trucks in Design Direction | | |

| Vehicle Class | Percent of ADT | Annual Growth | Initial Truck Factor | Annual Growth in Truck Factor | Accumulated 18-kip ESALs (millions) |
|---|---|---|---|---|---|

| | | |
|---|---|---|
| Total Calculated Cumulative ESALs | | million |

## Faulting

*Doweled*

| | | |
|---|---|---|
| Dowel Diameter | 1.5 | in |
| Drainage Coefficient | 1.00 | |
| Average Fault for Design Years with Design Inputs | **0.06** | in |
| Criteria Check **PASS** | | |

*Nondoweled*

| | | |
|---|---|---|
| Drainage Coefficient | 1 | |
| Average Fault for Design Years with Design Inputs | | in |
| Criteria Check | | |